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RIVERS UNDER INFLUENCE OF TERRESTRIAL ROTATION

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(Proc. Paper 1208)

SYNOPSIS

The article outlines the mathematical evaluation of forces involved in the formation of river beds, according to the Law of Baer. This Law states that on the northern hemisphere all rivers exercise pressure against the right bank, south of the equator against the left bank, affecting them more than the corresponding opposite bank. The mathematics of the article is based on the principle of dynamics regulating the Coriolis Force, which is created by a mass moving in a rotating system. In the given case the mass is the mass of river flow moving on the rotating earth's surface.

In a recent study of the hydrology of the Asiatic Continent, Professor Mietsch of Paris Sorbonne, discussing the morphology of giant Siberian Rivers, the Ob, Jenissei and Lena, draws attention to the formation of their beds due to the influence of the earth's rotation, in accordance with the theory formulated nearly a hundred years ago by the Baltic German scientist K. E. Von Baer. Also current Russian sources indicate that the "Law of Baer," as it is known among the geographers and geologists, is found in comprehensive Soviet hydrographic studies.

The origin of this theory goes back to 1860, when Von Baer submitted to the Petersburg Academy of Sciences, papers under the title, "Ueber ein allgemeines Gesetz in der Gestaltung der Flussbetten." This thesis states that in the northern hemisphere all rivers exert pressure against the right bank, and south of the equator against the left bank. Von Baer made an extensive study on the Volga River, which he found to be an outstanding example in support of this theory. The high elevated and eroded right bank of the Volga, which he calls "Bergufer," on which the settlements and villages are located, as contrasted to the "Wiesenufer," the depressed left bank, swampy

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and periodically flooded, which is unsuitable for dwellings, shows all the morphologic phenomena of his theory. Von Baer found similar conditions on the Don, Donetz, Dniepr and also on all the northward flowing rivers of Russia. He spent a great deal of his time in traveling in all parts of the world, surveying rivers such as the Nile, major rivers of India, China, North and South America, and the rivers of West and Central Europe, and drawing maps of them. Everywhere he found justification for his theory and in cases of exception, he tried to find reason for the deviation. Based on this theory, Von Baer drew some interesting conclusions. He reasoned that the right river bends are always sharper and more eroded than the left ones. Also the right arms of river deltas are usually the strongest. Furthermore, because the river progresses in the direction of the right bank, the left side of the river valley must show abandoned and dead water beds. He pointed in particular to the Nile, Rhine and Danube, to support his statement.

The mathematical background of the theory, which Von Baer who himself was a biologist by profession, tried to formulate and which he submitted in latter years to the French Academy, was received with skepticism on the part of mathematicians and physicists. He was told that the forces involved are too small to exercise any influence on the morphology of rivers, which is primarily a function of hydrologic and physiographic conditions in the drainage area. However, subsequently the Law of Baer was accepted by geographers and geologists as valid morphologic phenomenon and voluminous material based on current and historical cases was accumulated in support of this theory. A short bibliography at the end of this article shows to what extent the problem was discussed among the leading geomorphologists of the world.

The Law of Baer is not so well known among civil and hydraulic engineers. Sometimes it is discussed in textbooks on hydrology, however, in the professional literature on river regulation and hydraulic structures, where the evaluation of forces involved would be appropriate, it is hardly ever mentioned. The reason for this omission has to be attributed to the fact that the forces, affecting the river flow because of terrestrial rotation, are very small and the grasp of them not simple. The mathematical reasoning of the Law as submitted by Von Baer is unsatisfactory. Also the following attempts to establish the mathematical background of the Law, which would be comprehensible were not convincing.

In the period preceeding World War II, the Germans made comprehensive studies of Russian hydrography, mostly for military purposes. These were prepared by leading geographers, engineers and other scientists who brought the writings of Von Baer again under scrutiny. In 1942, Professor Dantscher of the University of Muenich, presented a new mathematical evaluation of Von Baer's river phenomena, based on the Coriolis Law of dynamics.

According to Corioli, in a rotating system, with an angular velocity "$\underline{u}$," a body moving with a linear velocity "$\underline{v}$" has two accelerations, primary or centrifugal acceleration "a_1" and a secondary or additional acceleration "a_2," known as Coriolis acceleration. The magnitude of the Coriolis acceleration is given by

$$a_2 = 2\,(\underline{v}\,\underline{u}) \tag{1}$$

in which "$\underline{v}$" and "$\underline{u}$" are the vectorial quantities of the linear velocity of the body and the angular velocity of the system. The term (v u) in equation (1)

indicates a vectorial product. The force produced by such motion of a body of mass "M," called Coriolis Force is then

$$C = M \cdot a_2 = 2 M (\underline{v}\ \underline{u}) \tag{2}$$

The actual value of "C" is then, according to the rule of vectors

$$C = 2 M \cdot v \cdot u \cdot \sin e \tag{3}$$

in which "v" and "u" are absolute quantities of linear and angular velocities and "e" is the angle between the pertinent vectors. The product reaches a maximum at e = 90 deg. (See Fig. 1.)

In the case of a river flowing with a linear velocity "v," the motion is in a system rotating with a terrestrial angular velocity "u." Considering the river surface as lying in a plane "E" tangential to the earth sphere at the point "A" (See Figures 2 and 3), we resolve the vector "$\underline{u}$," which is parallel to the earth's axis, into the components "$\underline{u}_1$" perpendicular to the plane "E" in the point "A" and "$\underline{u}_2$" located in the plane "E." For the time being let us disregard component "$\underline{u}_2$." The component

$$\underline{u}_1 = \underline{u} \cdot \sin f \tag{4}$$

indicates that the tangential plane "E" rotates with angular velocity "$\underline{u}_1$, while at the same time the river flows in the same plane with linear velocity "v." Both vectors "$\underline{u}_1$" and "$\underline{v}$" form together a right angle and consequently "e" in Equation 3 is 90 degrees and its sine is "1." The Coriolis Force is then

$$C = a M (\underline{v}\ \underline{u}_1) = 2 M \cdot v \cdot u \sin f \tag{5}$$

The magnitude of the diverting force is only a function of geographic latitude and equals twice the produce of the velocity of the stream; the angular velocity of the earth; the sine of pertinent geographic latitude and the mass of the flow. The direction of the stream does not enter into the calculation of the magnitude of this force. The direction of Coriolis acceleration can be determined in the following manner.

Considering the tangential plane "E" in the shape of a disc its absolute motion observed outside of the earth is counter-clockwise north of the equator and clockwise south of it. For an observer standing on the disc watching the flow of the river, the deviation due to the rotation will be toward the right on the northern hemisphere and toward the left on the southern hemisphere (See Figures 4 and 9). As a result, the rivers press on the right bank north of the equator and on the left bank south of it, regardless of the direction in which the river flows.

The angular velocity "u" in the Equation (5) is calculated from the time the earth requires for a complete rotation, which is 86,464 seconds and "u" is then 0.00007292 radians per second. Calculating the Coriolis Force for 1 cubic meter of flow, we have to express mass "M" in absolute metric units

$$M = 1000(\text{kg}) : 9.81(\text{m s}^{-2}) = 102(\text{kg m}^{-1}\text{s}^{2})$$

in which 9.81 meters per second stands for the gravity acceleration. The magnitude of the Coriolis Force is then, according to Equation (5)

$$C = 0.014292 \cdot v. \sin f \text{ (kg)} \tag{6}$$

As already mentioned the force resulting from the earth's rotation is very small, however, it does exist and may reach a considerable value when the flow has a high velocity at a high discharge rate, such as appears during flood stages. The following table indicates the values of this force for 1 cubic meter of flow at 1 meter per second velocity:

Nile River, at the origin,	0.0 Deg. alt.	0.0000 kg.
Nile delta,	30 " "	0.0071 "
Po River,	45 " "	0.0101 "
Upper Mississippi	45 " "	0.0101 "
Danube River, at Vienna	48 " "	0.0106 "
Northern German rivers		
Elbe, Rhine, Weser, Oder	53 " "	0.0114 "
Siberian Rivers		
Ob, Yenissei, Lena	60 " "	0.0124 "

For a rectangular cross section of a river "B" meter wide, and "T" meter deep, Dr. Dantscher evaluates the force acting on the right bani (See Fig. 5).

$$P_c = B \cdot 0.01429 \cdot v \cdot \sin f \quad (kg/m^2) \tag{7}$$

Assuming the relationship between "B" and "T" according to the empirical formula of Siedek

$$T = (0.0175 \cdot B - 0.0125)^{\frac{1}{2}}$$

corresponding values of "B" and "T" are

B = 200 m	T = 1.9 m
1000 m	4.2 m

For rivers at 60 degrees latitude and 3 m/sec velocity, the Coriolis pressure amounts to 8 and 40 kg/m^2 for 200 and 1000 m river width, respectively.

The effect of this force may become apparent in the branching of rivers, also in the formation of deltas where the right branch generally carries more flow than the left, provided that controlling conditions are the same. Sudden branching of rivers toward the right bank is a common phenomenon. Nogat river separates from the main stem of the Vistula River, south of Danzig, choosing its own course toward the sea. Also the formation of deltas of giant Siberian Rivers appear to be the result of this force.

A functional relationship between the Coriolis Force and transformation of the right bank is problematic and no attempts have been made to establish formulas on the pattern developed for the tractive effort of a stream and the volume of detritus carried as a result of stream bed changed by erosion.

According to the principles outlined by the French hydraulic engineer Fargue on the formation of river channels, a natural stream moves in sinusoidal curves and contracurves marking the greatest depths in bends "A" and

"B" and the least depth on crossings (fords) located in the point of inflection "C" (See Figure 6). Dr. Dantscher confirms the findings of Von Baer, that the river currents erode the right bank at "B" faster than the left bank at "A" resulting in a sharper curve in the right banks. In the case of a regulated river with structurally stabilized banks, any kind of revetment can resist the Coriolis Force because of its negligent value, however, the turbulence of the flow thus created attacks the bed, increasing its erosion. On the Rhine River, between Strassburg and Maxau, the difference in the eroded bed is 0.40 m between the right and left bank. On the Danube River at Vienna this difference amounts to 0.30 m. Both cases are based on actual measurements. This phenomenon can be generally accepted for any river, however, it does not establish any yardstick for determining the magnitude of the Coriolis Force.

An interesting suggestion for the evaluation of the Coriolis Force, based on rotational movement of icepacks, is given by Dr. Dantscher. The floating icepacks generally rotate clockwise on the right side of the river and counterclockwise on the left side, because of the higher velocity in the talweg. These icepacks react in a very sensitive way to all forces, even the smallest. Dantscher assumes that because of the Coriolis Force, the icepacks on the right side must rotate faster than those on the left side. The measurements could be made by a series of photographs. It is not known if the experiment was ever made.

Another application of the influence of the Coriolis Force on river flow is in the field of hydropower development. In the run-of-river power plants the power generating units are often placed toward the right bank because under equal conditions the turbine efficiency is higher there than when placed on the left side of the river. Also left turning turbines with vertical shafts have a higher degree of efficiency than right turning turbines. (In the northern hemisphere, see Figure 7)

The Coriolis Force is not the only result of terrestrial rotation interfering with the river flow. In Figure 3 the angular velocity "u" was resolved into components "$\underline{u}_1$" and "$\underline{u}_2$" which was disregarded. Its magnitude is

$$\underline{u}_2 = \underline{u} \cdot \cos f \tag{8}$$

Taking this component into consideration, the tangential plane "E" turns around the vector "$\underline{u}_2$," sinking in the east, and rising in the west. This component influences the flow of rivers located in the plane "E" only, when the river flows along the geographic parallel. A body moving east is seemingly loosing weight, while on the contrary, the west moving body is gaining weight, according to the law of dynamics. For the east-west moving rivers it is the same as if a larger gradient would be created for the west-east flowing rivers and they should dig deeper. However, these forces are so small that their effects even after centuries of duration could not be established.

It was previously indicated that the existence of the primary acceleration "a_1" produces a resulting contrifugal force. At latitude "f"

$$a_1 = r \cdot u^2 = R \cdot \cos f \cdot u^2 \tag{9}$$

The resulting force for a body of mass "M"

$$K = M \cdot a_1 = M \cdot R \cdot \cos f \cdot u^2 \tag{10}$$

in which "U" is the angular velocity of the earth and "R" radius of the terrestrial sphere. Resolving the force "K" into components "K_1" and "K_2"

$$K_1 = K \cdot \cos f = M \cdot R \cdot u^2 \cdot \cos^2 f \qquad (11)$$

which acts in the direction of R

$$K_2 = K \cdot \sin f = M \cdot R \cdot u^2 \cdot \sin f \cdot \cos f \qquad (12)$$

which acts in the direction of the meridian toward the equator. The calculation is made for spheroidal shape of the earth. The resultant "K" and idealized gravity "M . g" indicates the direction of actual gravity "M . g'" (See Figure 8). The perpendicular to "g'" in the point "A" is the tangent to earth geoid. The deviation is so small that the angle "f'" can be considered as equal to "f" and equation (12) is valid. This force disappears on the equator and on the poles. Its maximum is at "f" equal to 45 degrees, at which

$$K_2 = 0.5\, M \cdot R \cdot u^2 \qquad (13)$$

In the case of a north-south flowing river at 45 degrees latitude for 1 cubic meter of flow, "K_2" is equal to 1.69 kg, which is 0.169% of the gravity. It diminishes to 0.148 and 0.109% at 30 and 20 degrees latitude, respectively.

This force is considerably larger than the Coriolis Force. However, while the Coriolis Force appears on all rivers, flowing in any direction, the Force "K_2" should be considered for rivers flowing in meridional direction. This force acts as a gradient increase for rivers flowing from north to south, resulting in an increase of flow velocity and greater bed erosion. The contrary effect is found on rivers flowing in a south-north direction. This applies to the northern hemisphere. The correction would have to be taken for the southern hemisphere.

An example of the influence of the force "K_2" is found in the west Alps of Europe, a mountainous chain along the 40-45 degree latitude, north, at which the force has its maximum. The south flowing rivers of this region, the Tessin, Rhone, Adige, developed a very high gradient and consequently the river beds on southern slopes of the Alps are more eroded than the river beds on the northern side. Also the Mississippi and Missouri Rivers, west of the Great Lakes show a high gradient and are deeply cut in the earth's crust. The same may be applied to the southern slopes of the Himalaia Rivers of India. The component of terrestrial angular velocity "$\underline{u}_1$" explains also the formation of free vortex escape whirl observed on reservoir outlets in various parts of the world. They rotate counter-clockwise in the Northern and clockwise in the Southern hemisphere (See Figures 9 and 10 a and b).

Figure 10a shows a counter-clockwise vortex at the Loch Treig (Scotland) storage reservoir outlet tunnel. The hydraulic head above the tunnel outlet was 15 m. Fig. 10b, shows a clockwise vortex at the Arapuni hydropower development storage reservoir, when it was emptied through an 8 m. diameter diversion tunnel. The head over the tunnel outlet was 23.5 m. Arapuni is located in New Zealand.

Another outstanding example of the influence of terrestrial rotation and Coriolis force is the formation of currents in the landlocked-in seas and large lakes, fed by large inland streams. The current Soviet and satellite publications emphasize this phenomenon and demonstrate it on numerous examples. Fig. 11 a,b, show the formation of major currents on the Caspian Sea due to the Volga river and on the Black Sea due to Danube, Don and other smaller tributaries.

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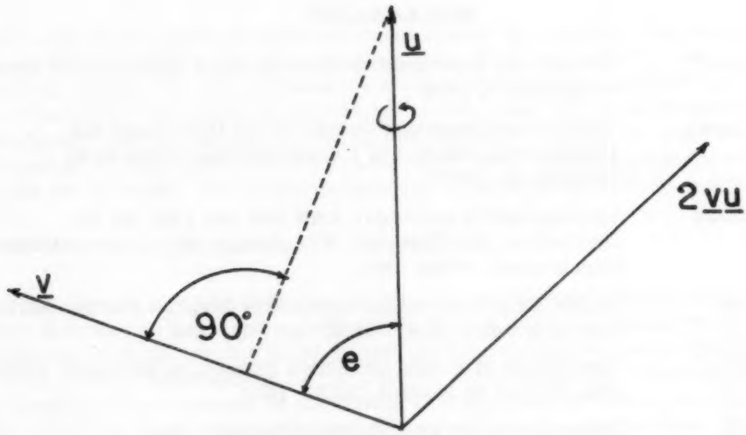


Fig. 1.

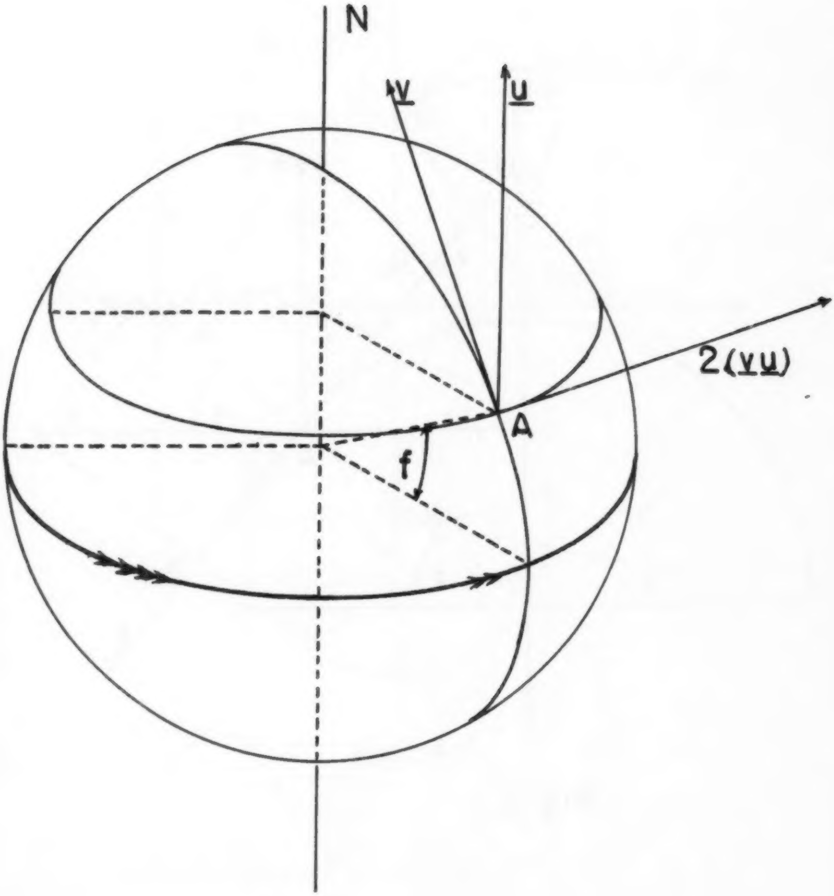
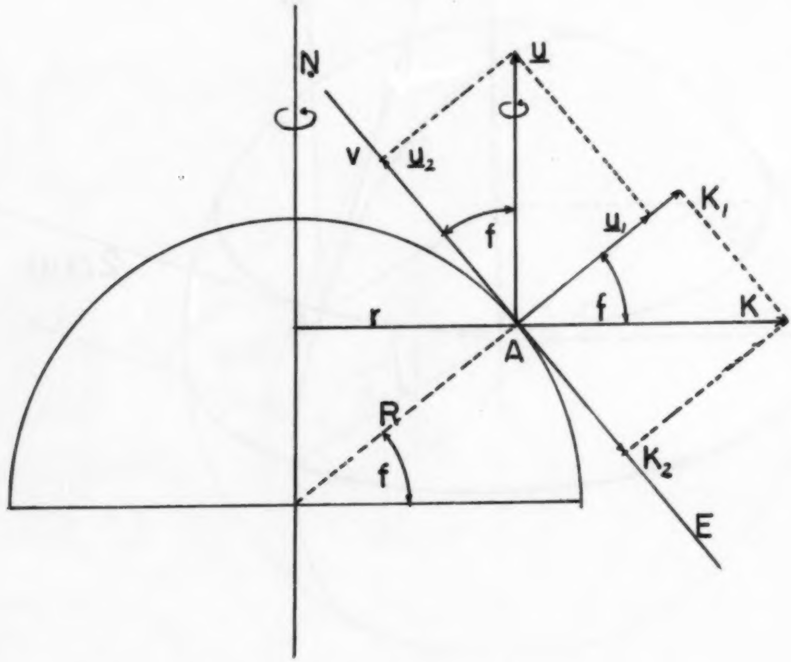


Fig. 2.



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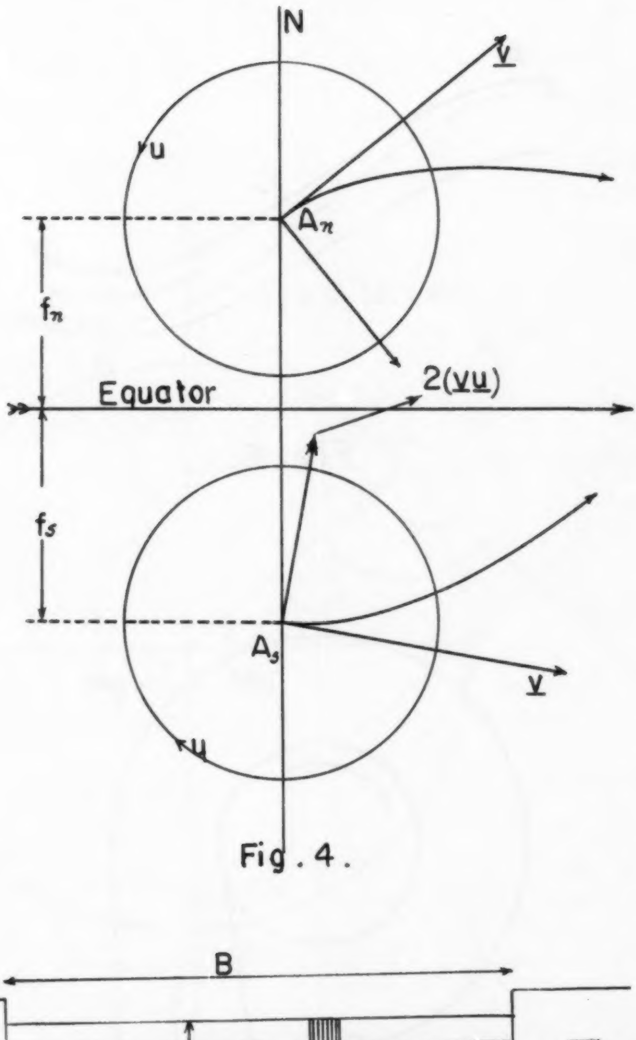

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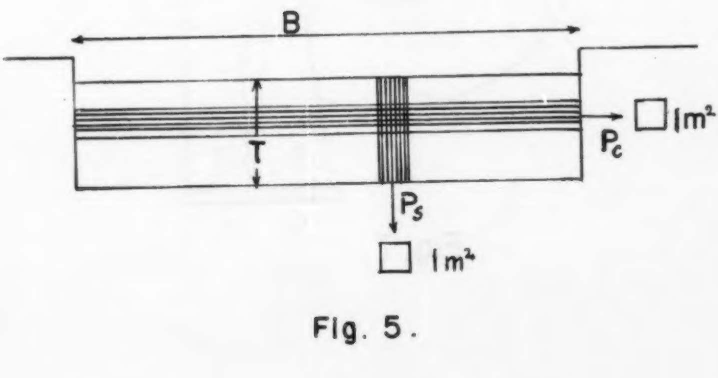

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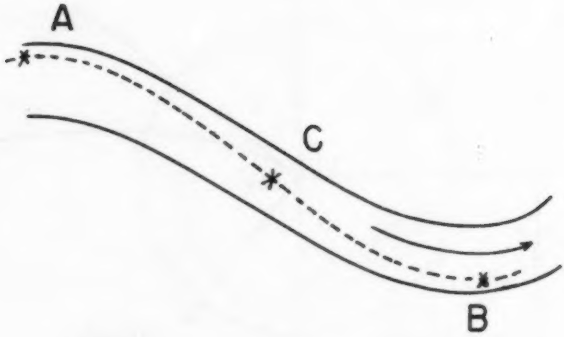

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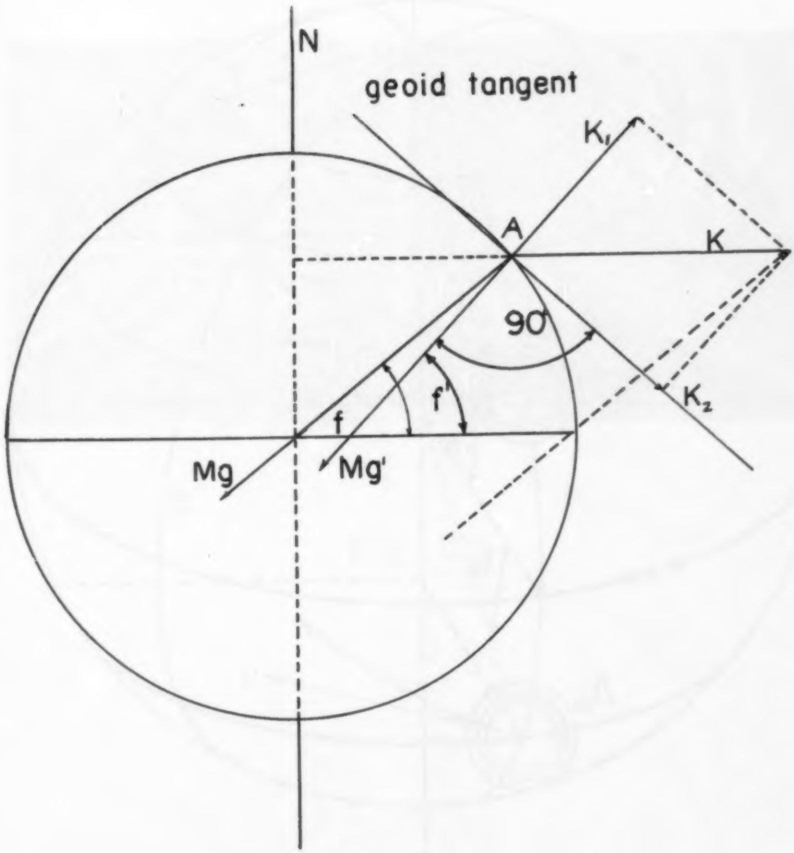
Fig. 7.



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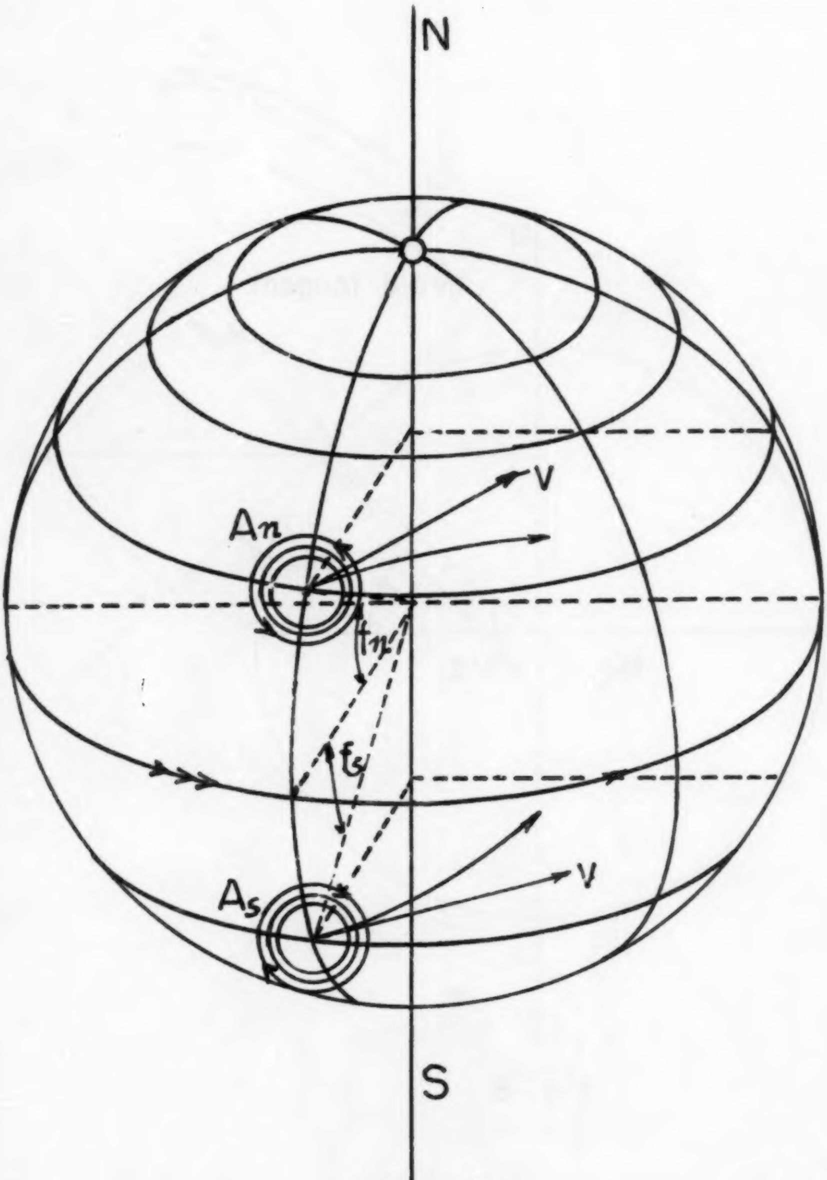
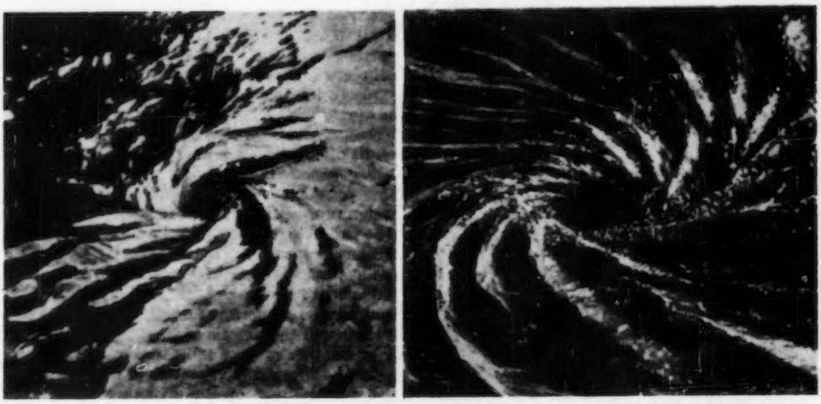


Fig. 9.



a. b.

Fig. 10.

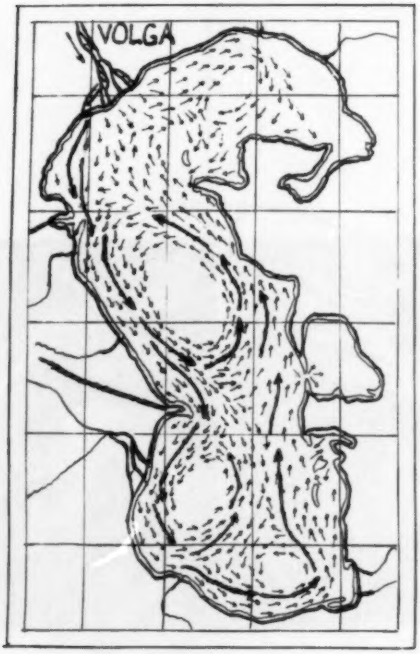


Fig. 11a.

Fig. 11b.

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FLOOD CONTROL PLAN FOR THE OHIO RIVER BASIN[a]

Bruce R. Gilcrest,[1] Emil P. Schuleen,[2] M. ASCE, and
Edgar W. Landenberger[3]
(Proc. Paper 1209)

SYNOPSIS

Floods in the Ohio River basin occur as the result of widespread weather disturbances and intense local rainfall. They are capable of causing tremendous flood damages. Because of the variety of physical conditions encountered in the basin all methods of flood control have application. A balanced, basin-wide program of reservoirs and local flood protection measures has been developed by the Corps of Engineers to reduce flood damages at principal damage centers and along thousands of miles of the basin's streams. As of 1956, a substantial portion of this program has been completed or is under construction, and progress is being made toward solution of the essentially independent flood problems of headwater and upstream urban areas. Water supply, low flow regulation, recreation, power, and conservation services are being provided in conjunction with flood control reservoir development. The accomplishments of completed works demonstrate the economic value of the program and its adaptability to the changing and expanding water resource development needs which are accompanying unprecedented expansion in basin activities. Vigorous prosecution of the plan in the service of these needs clearly is in the public interest.

Note: Discussion open until September 1, 1957. Paper 1209 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 83, No. WW 1, April, 1957.

a. Presented at a meeting of the Waterways and Harbors Division, Annual ASCE Convention, October 15-19, 1956, Pittsburgh, Pa.

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Flood Occurrence

Cyclonic weather disturbances moving easterly across the continent in winter and early spring are the principal cause of great Ohio River floods. They result in high tributary flows which combine to produce high stages on long reaches of the parent stream. Snow melt from northern and eastern basin areas; minimum infiltration, transpiration, and evaporation rates; and maximum runoff rates—all characteristic of that season—are contributing factors. Tributary flooding also may be caused by intense local rains which are much less seasonal in their occurrence. Ordinarily, however, storms of this nature are not sufficiently widespread to cause serious main-stem flooding.

Without benefit of the basin's flood control reservoir program, the Ohio River would crest above flood stage about once in 9 months at Pittsburgh, about once in 16 months at Cincinnati, and about once in 18 months in the lower valley. Flooding at many tributary locations is much more frequent.

A rough index of the severity of the flood problem is provided by comparison of maximum stages of record with flood stages. At ten typical large cities on the Ohio River the maximum stage of record has exceeded flood stage by an average of 22.4 feet, ranging from 18.8 feet at Evansville to as much as 29.1 feet at Louisville. In comparison, Corps of Engineers' flood emergency plans become effective when Ohio River flood stages are exceeded by as little as 1 to 5 feet, and major flooding occurs when they are exceeded by as little as 3 to 10 feet. The same stage comparison provides a rough measure of the severity of tributary flooding, although the range of values obtained is greater than on the main-stem. At twenty tributary locations selected at random maximum stages of record were found to have exceeded flood stages by an average of 15.4 feet. The range for individual locations was from 2.4 to 27.7 feet.

Substantially all major Ohio River floods, and most minor ones as well, have occurred during the months of December through April. At three typical main-stem locations (Pittsburgh, Cincinnati, and Evansville) the average distribution of maximum annual experienced during 68 years of record, by months, was as follows:

Month	Percent	Month	Percent
January	16	July	1
February	24	August	2
March	28	September	1
April	17	October	0
May	2	November	2
June	1	December	6

(December through April - 91 percent)

The extent to which tributary floods follow a marked seasonal distribution pattern varies roughly with drainage area and location. For many larger tributary areas the pattern is similar to that of the main-stem. Thus, at four tributary locations selected at random (Allegheny River at Franklin, Pa.; Hocking River at Athens, Ohio; Kanawha River at Kanawha Falls, W. Va.; and West Fork of White River at Noblesville, Ind.) the average distribution of maximum annual stages, by months, was as follows:

Month	Percent	Month	Percent
January	15	July	2
February	14	August	2
March	23	September	2
April	18	October	1
May	9	November	5
June	3	December	6

(December through April - 76 percent)

The periods of record used in developing the above tributary data ranged from 26 to 64 years, averaging 40 years. The drainage areas at the four selected points ranged from 837 to 8,367 square miles, averaging 4,030 square miles. On other areas, particularly smaller ones, local physical conditions and local rains may result in major departures from this distribution pattern. And, in general, large and maximum tributary floods are more likely to occur outside of the winter-early spring season than is the case on the main-stem.

Flood Damages

Floods on the Ohio River and its tributaries result in widespread damage to lands and improvements, cause loss of life, and disrupt orderly processes of commerce. Under 1956 conditions of development, but without flood control, average annual damages of about 125 million dollars could be expected in the basin's major damage centers, exclusive of the Tennessee River area.

The floods which produce such great damages range in size from the local nuisance variety which may be experienced several times each year at some places to those of disastrous proportion as experienced in 1913, 1936 and 1937 on large areas of the Ohio basin. The combinations of physical and economic conditions encountered are many, and produce flood damage patterns of wide variety, as indicated by the following examples:

During January and February 1937 the highest Ohio River stages of historical record occurred along the 716-mile reach from Point Pleasant, W. Va., to Cairo, Ill. All of its tributaries were in flood and record stages were reached on the lower reaches of those flowing northerly to the Ohio River in the area of the basin from Cincinnati to its mouth. Rainfall averaged 10 inches between January 6 and January 25, over the entire basin area of 203,900 square miles. Most telegraph, telephone, and power facilities along the Ohio were disrupted for periods up to a month, and rail service between north and south was interrupted at all points below Wheeling, W. Va. With the exception of the suspension bridge at Cincinnati, all highway bridge approaches in the 800-mile reach of river below Marietta were flooded. More than a half-million people were driven from their homes and business and industry were paralyzed. Recurrence of the 1937 flood under 1956 conditions of development, but without flood control measures in operation, would cause damages of 1.2 billion dollars or more.

The upper Wabash River provides a good example of a large tributary area flood problem involving both urban and extensive agricultural development. In the 317 mile reach between the White River and Huntington, Ind., the Wabash flood plain has an area of about 560 square miles, representing an average width of about 1.8 miles. Drainage areas at the upper and lower limit

of this reach are 708 and 16,440 square miles. As of 1956, lands and improvements subject to flooding include approximately the following:

Urban areas (exclusive of Delphi and Vincennes which have local flood protection):

Residences, number	6,900
Commercial properties, number	800
Industrial properties, number	33
Other properties (public buildings, etc.)	160
Value of property in flood plain	$133,000,000

Agricultural areas (including those afforded partial protection by completed agricultural levees):

Area, acres	358,000
Value of lands and improvements in flood plain	$61,000,000

Highways and railroads (including those afforded partial protection by agricultural levees):

Highway locations subject to flood, number	45
Value of highways in flood plain	$12,100,000
Railroad locations subject to flood, number	24
Value of railroads in flood plain	$24,100,000

Average annual flood damages of about $5,400,000 are to be expected in this area under 1956 conditions of economic development and with flood control measures in operation in 1956, distributed about as follows:

Agricultural damage:	
Crop	$2,590,000
Non-crop	1,960,000
Subtotal, agricultural damage	$4,550,000
Urban damage	665,000
Highway and railroad damage	185,000
Total	$5,400,000

Without these flood control measures average annual damages in excess of 7 million dollars could be expected. The magnitude and frequency of flood occurrences which produce these damages are indicated by Table 1.

TABLE 1. - FLOOD FREQUENCIES, WABASH RIVER BASIN

Location (Indiana)	Bankfull Stage Ft.	Bankfull Stage Occurrences per 100 yrs	Stages Equalled or Exceeded Once in 2 yrs	4 yrs	10 yrs	100 yrs
Wabash	12	450	19.7	21.1	22.6	26.2
Peru	20	28	-	20.4	22.8	26.8
Logansport	15	58	15.3	16.5	18.5	24.0
Lafayette	11	500	21.7	23.6	26.2	31.9
Clinton	18	350	24.0	25.4	27.3	30.9
Terre Haute	14	315	20.8	22.7	25.0	29.9

The flood problem on Brush Creek at Princeton, W. Va., is reasonably typical of urban headwater flooding in the Ohio basin. Damaging rises from this 43 square mile drainage area may occur as often as three times a year and at any season, although they are most frequent in late winter and early spring. Flooding is confined mainly to wide bottom land which is crossed by several highways and a railroad. Many commercial and industrial establishments have been built around and in the flood plain because of the limited area available for community expansion but normal growth has been restricted. Improvements subject to flood damage have a current value of about 9 million dollars. They include 375 residences, 48 commercial establishments, 10 industrial establishments, and the usual streets, utilities, and related improvements. Average annual flood damages under 1956 conditions of development may be expected to exceed $82,000.

Twin Creek is a small stream in north-central Kentucky. There are 145 farms in its 27 square mile drainage area. Land use is as shown in Table 2.

TABLE 2. - LAND USE, TWIN CREEK, KENTUCKY

Item	Total Basin		Flood Plain	
	Acres	%	Acres	%
Cropland	3,065	18	270	33
Meadow	-	-	237	29
Pasture	10,090	58	294	36
Woodland	2,303	13	-	-
Miscellaneous and idle	1,960	11	16	2
Total	17,418	100	817	100

The Twin Creek flood problem is reasonably typical of small headwater areas in agricultural use. Average annual damages under 1956 conditions amount to about $19,000 distributed as follows:

Damage to	Amount
Crops and pasture	$15,600
County roads	2,100
Minor fixed improvements	1,300
Total	$19,000

These damages occur as the result of floods, averaging about 3.7 occurrences per year.

Origin of the Flood Control Plan

Residents of the Ohio River basin have long been concerned with its flood problems. As early as 1808 some agricultural levees were built by Wabash basin landowners and, in later years, local initiative resulted in provision of partial flood protection at such localities as Portsmouth and Lawrenceburg on the Ohio. In 1912 the Pittsburgh Flood Commission reported the results of an extensive study of reservoir possibilities in the area above Pittsburgh, recognizing the desirability of multiple-use reservoirs. And, following the great flood of 1913 the Miami Conservancy District designed and built a comprehensive system of retarding reservoirs and channel works for flood

protection in the Miami River basin in Ohio. Flood control planning on a basin-wide basis began with the growing Federal interest in flood problems sparked by authorization of comprehensive river basin studies in 1927—the so-called "308" studies.

In 1934, under the Public Works Administration relief program, the Corps of Engineers began construction of Tygart Reservoir in the Monongahela River basin and a 14-reservoir system in the Muskingum River basin. In 1935 steps preliminary to construction of Bluestone Reservoir on the New River, a tributary of the Kanawha, were taken under the same program. Following this start, occurrence of the great flood of March 1936, adoption of a national flood control policy in the Flood Control Act of 22 June 1936, occurrence of the disastrous 1937 flood, intensive continuing study and investigation, and a series of legislative actions have led to adoption and substantial partial accomplishment of a comprehensive Federal program for flood control and allied purposes in the Ohio River basin.

This program, under jurisdiction of the Corps of Engineers, provides for construction of 81 reservoirs and local flood control measures at 242 locations. It was authorized by river and harbor and flood control legislation extending over a twenty-year period, beginning in 1935, and is subject to supplementation and modification as required to meet changing needs and conditions.

Nature of the Flood Control Plan

Formulation of the Ohio River basin flood control plan required (1) establishment of objectives, (2) analysis of methods, and (3) selection and sizing of individual projects.

The established objectives range from protection of essentially independent headwater areas such as the city of Massillon, Ohio, and agricultural areas along Barnett Creek in the Green River basin; through protection of contiguous rural and urban damage centers along many hundreds of miles of tributary streams and the full length of the Ohio main-stem; to control of Ohio River discharge in the interest of reducing Mississippi River flood stages. In addition they contemplate provision of related services such as low flow regulation, direct water supply, recreation, power production, and major outlets for land drainage.

All methods of flood control have usefulness in the Ohio River basin, but in varying degree as to size and area of application. Main-stem reservoirs, which would be most effective in contributing to reduction of major Mississippi River floods, are out of the question because of extensive development of the valley. Similarly, the development of many major tributary valleys makes reservoir construction undesirable. Channel improvement is impractical on the main-stem and many tributaries because of the extreme range of stages encountered but has widespread application on smaller streams in the basin. Levees and flood walls have application at many urban centers where flood damages are concentrated. Levees can be used effectively in agricultural areas where the flood plains are broad and large areas are inundated, but generally are too costly to be justified for the protection of narrow agricultural flood plains. Land treatment measures also have application in the basin, particularly in upland areas.

Units of the plan were selected and sized in recognition of the factors

TABLE 3. - DISTRIBUTION AND INTERRELATION OF ELEMENTS, OHIO RIVER BASIN FLOOD CONTROL PLAN

Basin	Reservoirs: Number (a)	Reservoirs: Net Drainage Area Controlled: Sq. Miles	Reservoirs: Net Drainage Area Controlled: % of Basin Area	Reservoirs: Miles of Stream to be Afforded Protection by Reservoirs (c)	Local Protection Projects: Total Number of Locations	Local Protection Projects: Number of Locations below Reservoirs
Allegheny R.	9	6,045	52	475	20	9
Monongahela R.	4	2,040	28	275	2	-
Beaver R.	4	1,035	33	145	-	-
Short Cr.	-	-	-	-	2	-
Muskingum R.	17	5,535	69	495	4	1
Little Kanawha R.	3	575	25	125	-	-
Hocking R.	2	120	10	95	-	-
Kanawha R.	7	8,010	65	460	1	-
Guyandot R.	1	270	16	30	-	-
Twelvepole Cr.	1	140	31	40	-	-
Big Sandy R.	4	855	20	180	-	-
Scioto R.	5	1,795	28	260	-	-
Little Miami R.	2	575	33	75	-	-
Licking R.	2	2,285	62	175	2	1
Mill Cr.	1	30	18	20	-	-
Miami R.	2	1,080	20	45	-	-
Kentucky R.	3	4,620	67	300	2	1
Salt R.	-	-	-	-	1	-
Green R.	5	6,630	72	435	-	-
Wabash R.	2	545	2	460	51	34
Saline R.	-	-	-	-	1	-
Tradewater R.	-	-	-	-	1	-
Cumberland R.	7	17,400	98	525	5	-
Tennessee R.	-	-	-	-	2	-
Cache R.	-	-	-	-	1	-
Subtotal Tributaries	81	59,585	36(b)	4,615	95	46
Ohio Main-Stem	-	-	-	980	147	147
Total	81	59,585	36(b)	5,595	242	193

(a) Exclusive of Tennessee Valley Authority reservoir program and reservoirs provided by local initiative.
(b) Based on total basin area of 203,900 sq. mi., less 40,600 sq. mi. in Tennessee basin or a net area of 163,300 sq. mi.
(c) Stream reaches benefited by more than one reservoir are not duplicated.

outlined above. Along the extreme upper Ohio main-stem, principal reliance is placed in reservoirs on tributaries of varying size. Below the Pennsylvania-Ohio line tributary reservoirs must be supplemented by local protection works at certain major damage centers to provide effective flood control. The tributary reservoirs which serve the main-stem also provide local protection. And, in basins like the Muskingum, principal reliance for tributary flood control is placed upon them. In other tributary basins, such as the Wabash, local works for protection of urban and rural areas are of primary importance, with limited supplemental control being provided by reservoirs. The locations of most of the local protection works in the plan will be afforded supplemental protection by reservoirs and this is recognized in the design of those local works. The distribution and interrelation of elements of the plan are indicated by Table 3.

Status of the Flood Control Plan

The 1956 status of the Ohio River basin flood control program is as given in Table 4.

TABLE 4. - STATUS OF OHIO RIVER BASIN FLOOD CONTROL PLAN

Project Status in 1956	Number	
	Reservoirs	Local Protection Works
Projects authorized	81	242
Projects complete or essentially complete	33	43
Projects under construction	3	13
Remaining authorized projects:		
Active (a)	14	45
Deferred (b)	27	35
Inactive (c)	4	106
Total	45	186

(a) Suitable for early construction.
(b) Require additional study to determine whether they are economically justified or desired under current conditions.
(c) Not economically justified under current conditions or, in case of some local protection works, are not desired by local interests at this time.

The distribution and interrelation of flood control works complete or under construction in 1956 are indicated by Table 5.

The degree of reservoir control at various locations along the Ohio River main stem is indicated by Fig. 1. The control effective in 1956 ranges from 15 percent at Smithland, Kentucky, and Evansville, Indiana, to 36 percent at Marietta, Ohio. Under the proposed plan the control would range from 36 percent at Wheeling, W. Va., to 52 percent at Marietta. The data shown for Paducah include the drainage area and reservoir control in the Tennessee River basin.

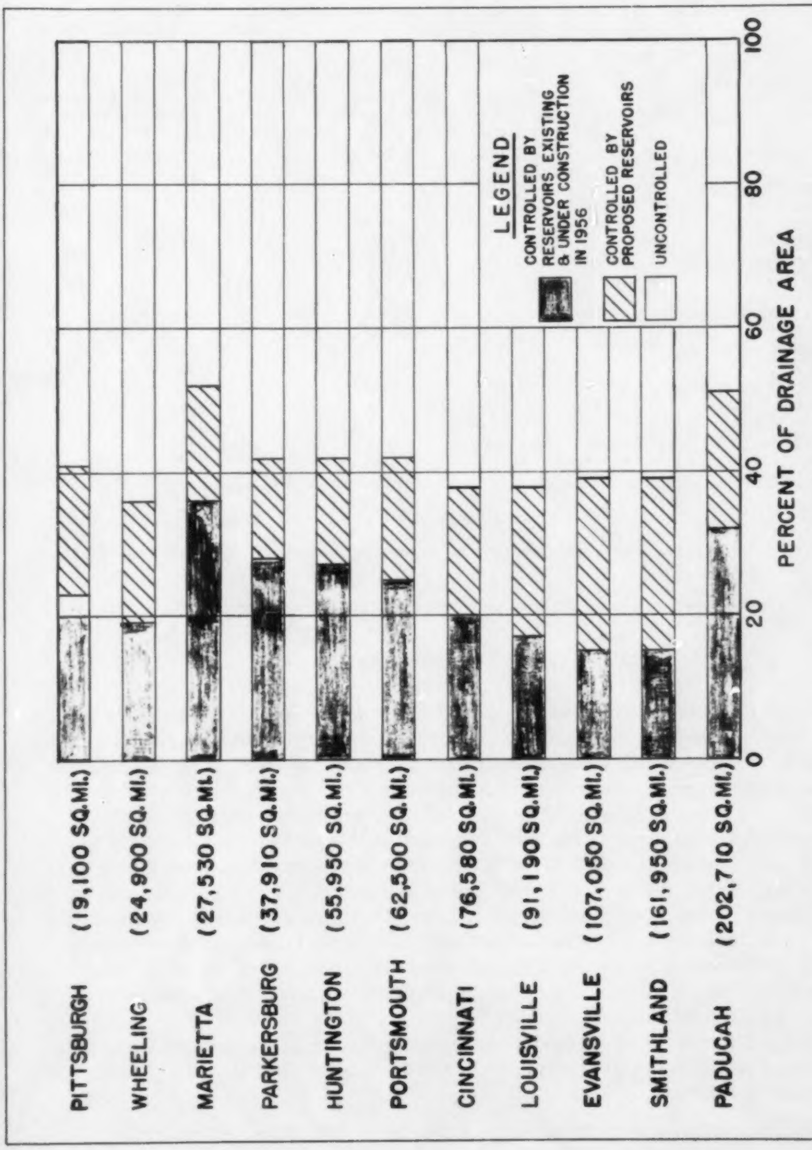

FIG. 1-DEGREE OF RESERVOIR CONTROL AT VARIOUS OHIO RIVER LOCATIONS

TABLE 5. - DISTRIBUTION AND INTERRELATION OF OHIO RIVER BASIN FLOOD CONTROL WORKS COMPLETE OR UNDER CONSTRUCTION IN 1956

Basin	Reservoirs					Local Protection Projects		
	Number (a)	Net Drainage Area Controlled: Sq. Miles	Percent of Basin Area	Percent of Authorized Control	Miles of Stream Afforded Protection by Reservoirs (c)	Total Number of Locations	Percent of Total Authorized Locations	Number of Locations below Reservoirs
Allegheny R	6	2,805	24	46	365	9	40	2
Monongahela R	2	1,620	22	79	230	1	50	-
Beaver R	2	345	11	33	140	-	-	-
Muskingum R	15	5,015	62	90	450	2	50	-
Hocking R	1	35	3	30	65	-	-	-
Kanawha R	2	3,970	32	49	265	1	100	-
Big Sandy R	1	205	5	25	80	-	-	-
Scioto R	1	380	6	21	160	-	-	-
Mill Cr	1	30	18	100	20	-	-	-
Kentucky R	-	-	-	-	-	1	50	-
Salt R	-	-	-	-	-	1	100	-
Green R	1	450	5	7	160	-	-	-
Wabash R	1	295	1	50	195	9	18	2
Saline R	-	-	-	-	-	1	100	-
Cumberland R	3	8,940	50	51	495	3	40	-
Cache R	-	-	-	-	-	1	100	-
Ohio Main-stem	-	-	-	-	-	27	18	27
Total	36	24,090	(b) 15	40(b)	2,625	56	22	31

(a) Exclusive of Tennessee Basin reservoirs under jurisdiction of the Tennessee Valley Authority.
(b) Based on area exclusive of Tennessee Basin.
(c) Stream reaches benefited by more than one reservoir are not duplicated.

Performance of Completed Works

As of 1956, the periods during which the various completed local protection works have been available for effective use vary from several months to about 18 years. While individual projects distributed over an area as large as the Ohio basin cannot be expected to exhibit the same degree of accomplishment during such a short period, a sufficient number were completed prior to and during the early nineteen-forties to demonstrate, by their performance on several major floods, the long range economic promise of the program. First costs—both Federal and local—for the completed local works total about 140 million dollars. During the 18 years since completion of the first unit, they have prevented about 110 million dollars in flood damages. Thus, on the basis of an average period of effectiveness of about 10 years the completed local projects have returned benefits equivalent to almost 80 percent of their first cost.

The performance of completed reservoirs in the prevention of flood damages also has been impressive. For convenience in appraisal they have been grouped as follows:

Upper river group consisting of 8 reservoirs above Pittsburgh and 2 in the Beaver basin.

Middle river group consisting of 19 reservoirs in tributary basins between Wheeling and Cincinnati, 14 of which are in the Muskingum basin.

Lower river group consisting of 4 reservoirs below Cincinnati, 3 of which are multiple-purpose projects in the Cumberland basin.

First costs and flood control benefits effected to 1956 by these reservoir groups are as given in Table 6.

TABLE 6. - FIRST COSTS AND FLOOD CONTROL BENEFITS FOR COMPLETED OHIO RIVER BASIN RESERVOIRS

Reservoir Group	First Cost (Rounded)	Flood Control Benefits to Fall of 1956 (Rounded)
Upper River	$115,000,000	$231,000,000
Middle River	90,000,000	78,000,000
Lower River	46,000,000(a)	12,000,000
Total	$251,000,000	$321,000,000

(a) Exclusive of costs allocated to power functions of Cumberland basin reservoirs.

Thus, the upper river group has returned flood control benefits equivalent to twice the first cost of the reservoirs in the group, and the middle and lower river groups have returned flood control benefits equivalent to about 87 and 26 percent, respectively, of the portions of their first cost chargeable to flood control.

In producing the benefits cited above, reservoir effects have ranged from substantially complete control of floods at tributary locations immediately below the reservoirs to a more moderate degree of control at distant downstream points. The estimated effect of reservoirs in operation at the time of recent floods is shown in Table 7.

The degree of protection afforded by completed local works also varies through a wide range, from full protection against maximum floods of record in most of the urban areas to protection against much more moderate floods in agricultural areas.

A valuable additional function of a number of the completed reservoirs—particularly those in the upper river group—is low flow regulation. To a large degree this service is possible because of the marked seasonal pattern of flood occurrence in the Ohio basin which permits retention of a portion of the characteristically high spring runoff for use in subsequent low flow months. Table 8 shows the storage allocations in effect at reservoirs used for low flow regulation, water supply, and power production in addition to flood control.

Substantial increases in low flow have been effected by reservoir operation. At Pittsburgh, for example, flow in the Ohio averaged only 2,800 c.f.s. during the driest 5-day period in October and November of 1953, of which 1,200 c.f.s., or 43 percent, was from reservoir storage. Substantially larger proportions of tributary flow were provided from reservoir storage during those months, averaging over 11 times the natural flow of the Monongahela River at Charleroi, Pa.; over 4 times the natural flow of the Mahoning River at Youngstown, Ohio; and over 8 times the natural flow of the Cumberland River at Nashville, Tenn. The large increase at Nashville was incidental to power generation. Without the low flow regulation provided on the Monongahela

TABLE 7 - ESTIMATED EFFECT OF OHIO RIVER BASIN RESERVOIRS IN OPERATION AT TIME OF RECENT FLOODS

Location	Flood Stage Feet	Dec.1942 - Jan.1943 Natural Crest(a)	Dec.1942 - Jan.1943 Reduction in Feet	March 1945 Natural Crest(a)	March 1945 Reduction in Feet	April 1948 Natural Crest(a)	April 1948 Reduction in Feet	Jan. - Feb. 1950 Natural Crest(a)	Jan. - Feb. 1950 Reduction in Feet
OHIO RIVER									
Pittsburgh, Pa.	25	14.4	2.8	10.2	1.8	8.1	3.3	-2.0	1.6
Wheeling, W. Va.	36	18.5	3.0	13.6	2.3	12.9	4.7	-4.0	2.2
Marietta, O.	35	16.7	3.0	16.2	3.0	18.4	6.3	-5.3	1.3
Pomeroy, O.	41	17.9	2.6	16.7	3.0	20.7	5.5	6.6	1.3
Huntington, W. Va.	50	11.7	1.6	11.4	1.5	14.6	2.9	7.6	1.0
Cincinnati, Ohio	52	9.8	1.0	17.7	0.5	13.2	0.4	7.2	0.2
ALLEGHENY RIVER									
Franklin, Pa.	17	2.6	1.4	-0.4	0.8	6.5	0.7	-3.4	1.0
Lock 7, Pa.	21	5.2	0.7	2.8	1.0	-0.6	0.9	-0.8	0.8
MONONGAHELA RIVER									
Lock 7, Pa.	21	1.6	1.0	1.0	0.5	4.4	2.3	1.7	1.5
Lock 4, Pa.	24	6.2	0.8	4.9	0.6	7.6	2.3	4.6	2.0
MAHONING RIVER									
Youngstown, Ohio	12	3.2	1.2	-0.4	2.7	-2.3	2.5	1.4	2.2
MUSKINGUM RIVER									
Coshocton, Ohio	17	0.1	4.8	1.4	6.1	1.8	6.8	-1.6	4.0
Zanesville, Ohio	25	0.4	3.1	6.1	2.5	5.4	6.8	-0.3	1.0
KANAWHA RIVER									
Kanawha Falls, W. Va.	25		(b)		(b)		(b)	-6.0	1.9
Charleston, W. Va.	34		(b)		(b)		(b)	-1.1	2.5
SCIOTO RIVER									
Columbus, Ohio	22		(b)		(b)		(b)	0.4	0.4
CUMBERLAND RIVER									
Celina, Tenn.	40		(b)	2.2	4.5	-9.2	2.9	13.3	1.0
Carthage, Tenn.	40		(b)	4.4	3.5	-10.0	2.4	13.3	8.1
Nashville, Tenn.	40		(b)	6.6	2.0	-7.9	2.3	11.9	3.3
Number of Reservoirs in Operation at Time of Flood		19		22		23		26	

(a) Natural crests measured in feet above flood stage.
(b) Reservoirs affecting this location not in operation.

TABLE 8. - STORAGE ALLOCATIONS FOR LOW FLOW REGULATION, WATER SUPPLY, AND POWER PRODUCTION, IN COMPLETED OHIO RIVER BASIN RESERVOIRS

Reservoir	Storage Allocation - Acre Feet							
			Portion of Net Usable Capacity Used for					
	Total Capacity	Net Usable Capacity	Flood Control (a)		Low Flow Regulation		Direct Water Supply	
			Maximum	Minimum	Maximum	Minimum	Maximum	Minimum
Tygart	289,600	273,400	278,400	178,400	100,000	-	-	-
Youghiogheny	254,000	243,800	151,000	99,500	149,300	97,800	-	-
East Branch Clarion	84,300	83,300	38,700	19,000	64,300	44,600	-	-
Berlin	91,200	89,400	55,800	32,800	56,600	33,600	(b)	(b)
Mosquito Creek	104,100	102,100	33,000	21,700	69,400	58,100	11,000	11,000
Delaware	132,000	123,600	123,600	118,000	5,600	-	-	-
Dewey	93,300	31,000	31,000	76,100	4,900	-	-	-
Reservoirs Used for Power Production			Minimum Dependable Flood Control		Maximum Drawdown for Power		Minimum Pool	
Wolf Creek	6,089,000	-	2,094,000		2,142,000		1,853,000	
Dale Hollow	1,706,000	-	353,000		496,000		857,000	
Center Hill	2,092,000	-	762,000		492,000		838,000	

(a) Maximum reservation effective winter and early spring and minimum reservation effective summer and fall.
(b) Ultimate storage reservation of 19,400 acre-feet is anticipated.

River, stream flows would have been deficient for purposes of navigation for 98 days and 43 days, respectively, on the upper and lower river during the months of August through November 1953. At Columbus, Ohio, a potentially serious domestic water supply shortage was averted by flow regulation from Delaware Reservoir. The aggregate value of low flow regulation to date—based primarily on the alternative annual cost of facilities required in its absence—is about 35 million dollars. Power revenues through June 1955 have amounted to about 14-1/2 million dollars.

An indication of the over-all economic performance of completed projects is provided by Fig. 2, which compares accumulated annual project costs with accumulated annual benefits. The costs shown include interest and amortization charges on Federal and local investments, and operation, maintenance, and replacement costs. The benefits shown include evaluated flood control, power, and low flow regulation benefits. The graph demonstrates clearly that on an annual cost—annual benefit basis the program is paying its way by a substantial margin. The excess of accumulated annual benefits over accumulated annual charges as of 1956, is about 370 million dollars—providing a benefit-cost ratio for all completed projects of about 3.1 to one.

Reservoir Cost Experience

Fig. 3 compares adjusted unit storage costs with gross reservoir capacities for the 33 completed reservoirs in the Ohio basin program. They were derived by escalation of actual reservoir costs by Engineering News Record Construction Cost Index ratios to provide a sound basis of comparison. In

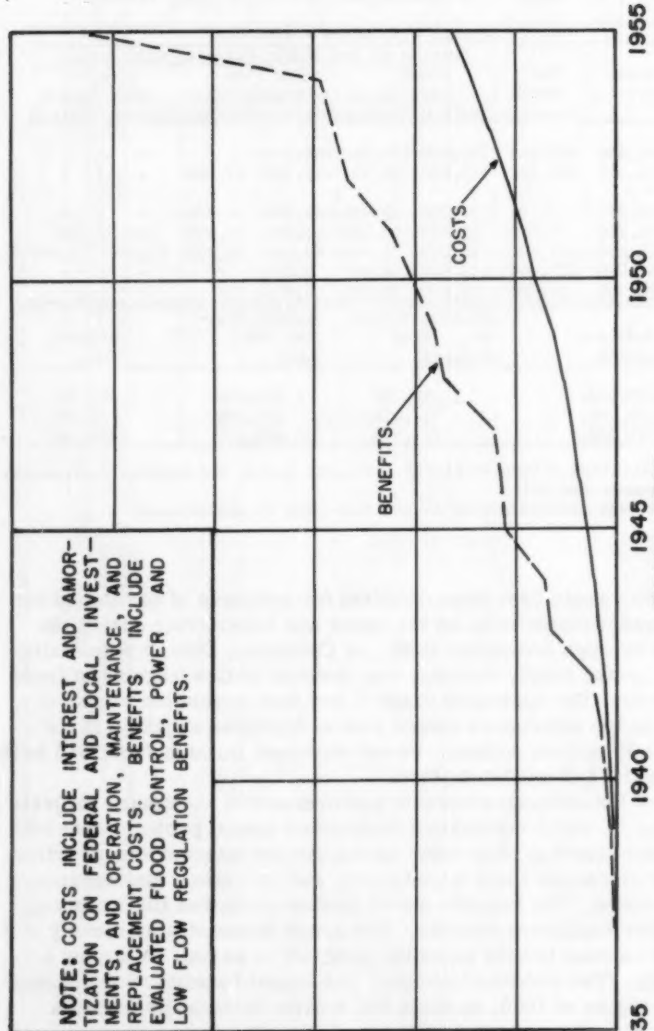

FIG. 2—COMPLETED PROJECTS—OHIO RIVER BASIN ACCUMULATED COSTS VS ACCUMULATED BENEFITS

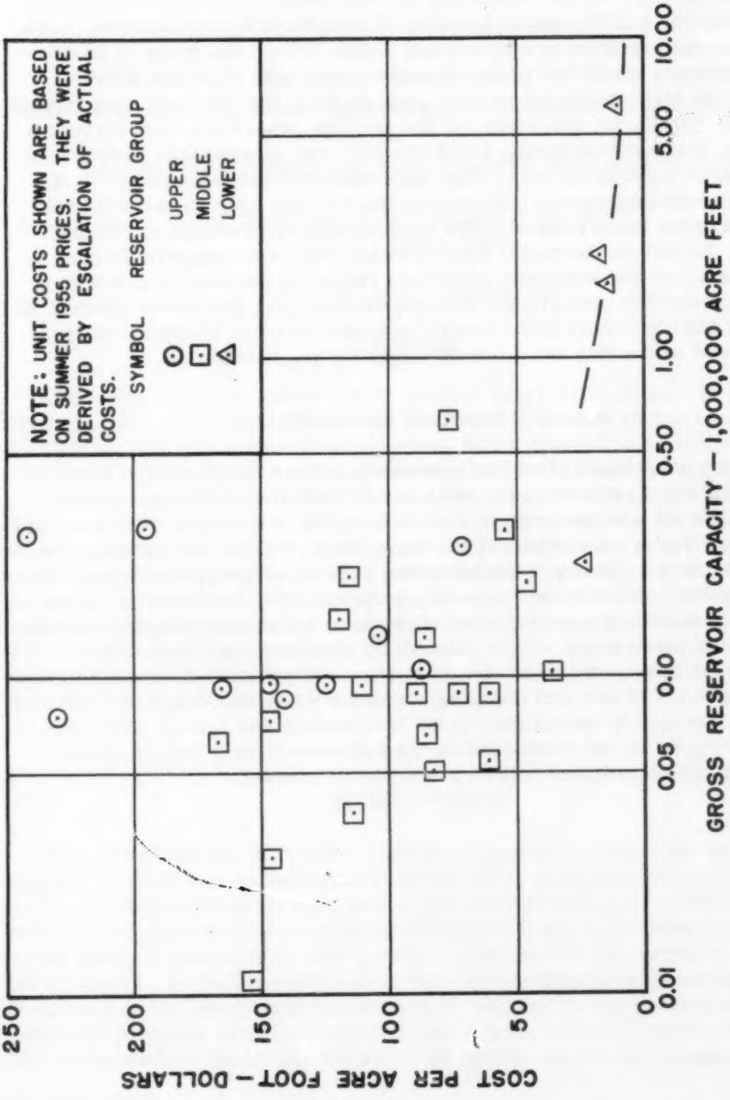

FIG. 3—COMPLETED RESERVOIRS — OHIO RIVER BASIN
UNIT STORAGE COST VS GROSS CAPACITY

addition, the separable cost of power facilities was eliminated in establishing adjusted unit costs for the three flood control-power reservoirs in the Cumberland basin.

With the exception of the relatively low unit costs applicable to the three largest reservoirs in the group, the size of completed Ohio basin reservoirs bears no marked relation to unit storage costs. Within the group of 29 completed reservoirs which fall in the capacity range from 12,000 to 290,000 acre-feet, the highest and lowest unit costs apply to the third and fifth largest reservoirs. Similarly, unit costs for the smaller reservoirs vary through a wide range, the smallest having a unit storage cost substantially higher than the 29 reservoir group average. The applicability of this experience to the entire reservoir program is indicated by the fact that only 23 percent of the reservoirs in the basin program will have capacities in excess of 300,000 acre-feet. Insofar as the major portion of the reservoir program is concerned it is clear that economic advantage cannot be associated with any particular reservoir size group. This experience is of particular interest in the light of the big versus little reservoir controversy with which most people concerned with water resource development are familiar.

Future of the Flood Control Plan

Although a substantial start has been made in the control of Ohio basin floods, much remains to be done. In terms of reduction in average annual flood damages the works complete and under construction as of 1956 provide only about a 50 percent solution of the basin flood problem. In addition, the rapidly expanding economy of the basin has been accompanied by expanding needs for water resources development, particularly in the fields of direct water supply and low flow regulation. A dynamic continuing program for the satisfaction of these needs will be insured by objective pre-construction planning of the authorized units of the program not yet started, and similarly objective analysis of new and changing problems which the Corps of Engineers is called upon to investigate in its examination and survey work. Experience provided by the completed works indicates clearly that vigorous prosecution of that program will be in the public interest.

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LABORATORY STUDY OF WIND TIDES IN SHALLOW WATER

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(Proc. Paper 1210)

SYNOPSIS

Wind tides and wave conditions in shallow water were studied in a laboratory channel. The experiments were conducted with smooth and rough bottom conditions, and with strips of cheese cloth in the channel to simulate the roughness effects of vegetation in nature. The results indicate a rapidly increasing set-up when the still-water depth decreases below a certain limit. There were no indications that the bottom roughness affects the set-up for relatively deep water; in very shallow water, however, the rougher bottom conditions result in higher set-ups. The trend is especially pronounced for higher wind velocities. For the shallowest still-water depth (0.05 foot) used in the experiments, the setup was approximately 10 per cent higher for the rough bottom, and approximately 20 per cent higher when strips of cheese cloth were used in the channel to simulate the roughness effect of vegetation, than the set-up observed with a smooth bottom.

INTRODUCTION

When wind blows over a water surface it generates waves, the heights and periods of which are a function of wind intensity, wind duration, depth, and fetch. In addition to generating waves, the wind produces a tangential stress at the water surface with a resulting surface current in the general direction of the wind. In deep water, this current is balanced by the backflow in the lower layers. In shallow water, however, the backflow is affected by the roughness of the bottom and the water will "set-up" to the leeward until a sufficient pressure head is reached to balance the effect of bottom roughness. The more shallow the water, the more the bottom affects the backflow of

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water, and the higher the relative set-up must be to create an equilibrium condition between the wind generated current on the surface and the backflow along the bottom.

In limited bodies of water such as lakes, bays, and seas, wind tides have been measured and found to vary from a few inches for light winds to many feet in extreme cases involving hurricane winds over shallow bodies of water. Wind tides are capable of creating vast destruction when levees and dikes are not sufficient to withstand the combined action of wind tides and waves. For the proper design of shore protecting structures, the design engineer must be able to predict the most critical condition for a given locality. The problem has been treated by numerous investigators theoretically as well as empirically. Wind tides and water-surface slopes have been measured and analyzed in nature in the Baltic by Palmen(1) and Palmen and Laurila,(2) and in numerous inland lakes by Hellstrom.(3) The most complete data available for a large variety of conditions is the Lake Okeechobee data, obtained by the Corps of Engineers, U. S. Army.(4) The latter data have been used by numerous investigators such as Hellstrom,(3) Langhaar,(5) and Saville,(6) to develop or verify various theories concerning wind tides. Small scale laboratory investigations have been completed by Keulegan,(7) Hellstrom(3) and Francis.(9) Dorn(10) has made a study in a large outdoor pond. Keulegan and Dorn, in their studies, demonstrated (by eliminating waves in some tests by the use of detergents) that two effects of the wind are involved: the surface traction of the wind, and the form resistance of the waves.

All of the laboratory investigations were completed, insofar as is known with a smooth bottom condition. For the natural bodies of water, however, the roughness of the bottom may have varied considerably; perhaps from being smooth sand on some occasions to being covered with dense vegetation on others. To study the effect of bottom roughness upon the characteristics of wind tides and the generation of waves, the experiments discussed below were completed using three different conditions of bottom roughness, i.e., (a) smooth bottom, (b) rough bottom, and (c) rough bottom with strips of cheese cloth in the channel to simulate the effect of the roughness of vegetation in nature. Each of the conditions of roughness were combined in several depths of water.

Definitions

The definitions used in this paper are shown in Figure 1.

F the fetch in feet; the distance from the leeward still-water shore line to the point the set-up was measured.

H wave height in feet.

MWL the mean-water-level (see Figure 1).

N the planform factor which takes the converging or diverging planform of the lake or channel into consideration.

S the difference in windward and leeward water-surface elevations in feet, when the bottom at the windward end <u>is not exposed</u>.

S' the elevation of the MWL at the leeward shore above the horizontal bottom when the bottom at the windward end <u>is exposed</u>.

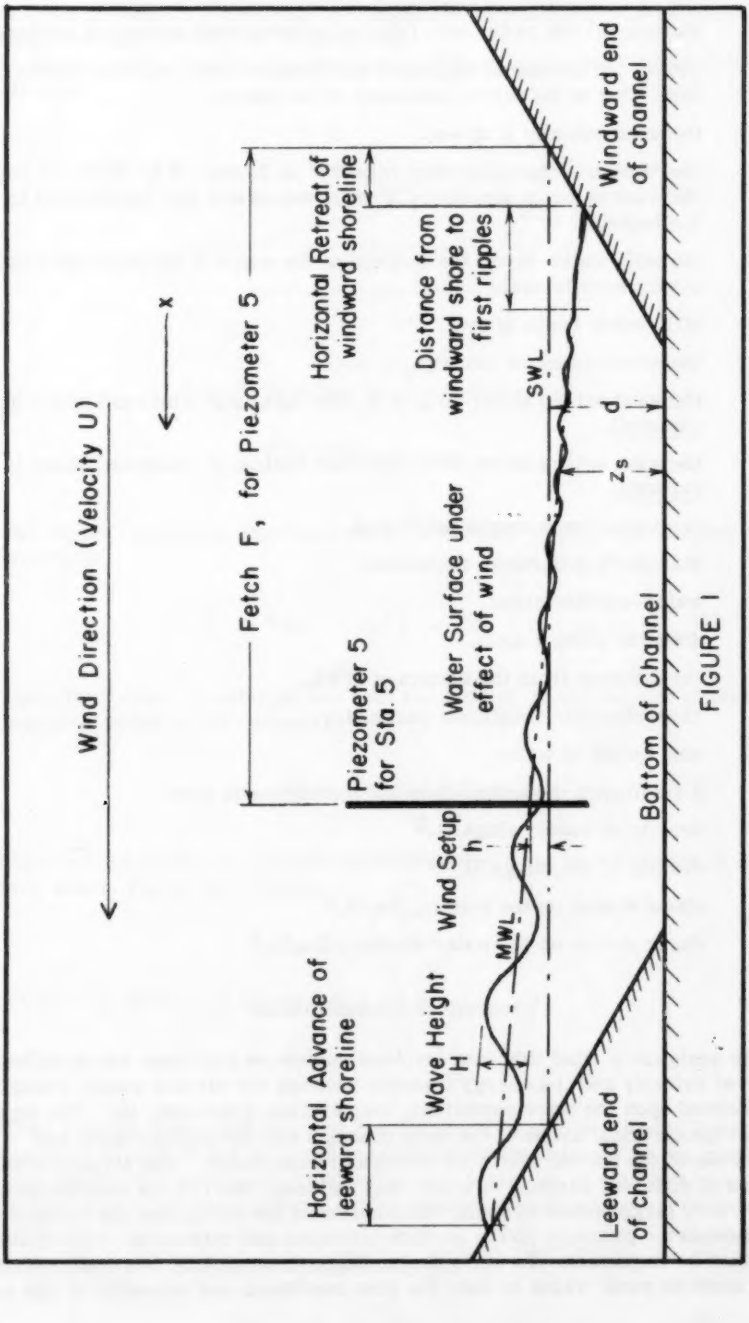
Wind Direction (Velocity U)
x
Fetch F, for Piezometer 5
Horizontal Retreat of windward shoreline
Distance from windward shore to first ripples
Windward end of channel
SWL
d
Zs
Water Surface under effect of wind
Piezometer 5 for Sta. 5
Wind Setup h
MWL
Wave Height H
Horizontal Advance of leeward shoreline
Leeward end of channel
Bottom of Channel

FIGURE I

S_1 set-up (difference in windward and leeward water-surface elevations) due to the skin friction between wind and water surface.

S_2 set-up (difference in windward and leeward water surface elevations) due to the form resistance of the waves.

U the wind velocity in ft/sec.

U_o the "formula characteristic velocity" in ft/sec. It is about 1.3 times the wind velocity necessary to start waves and was introducted by Keulegan.(7)

SWL the still-water-level; the surface of the water if all wave and wind action were to cease.

d still-water depth in feet.

g the acceleration of gravity.

h the wind set-up above SWL in ft. (the bottom at windward shore not exposed).

h' the wind set-up above SWL in ft. (the bottom at windward shore is exposed).

k_s equivalent sand roughness in feet.

n Manning's roughness coefficient.

s water-surface slope.

z distance along z-axis.

z_s the distance from the bottom of MWL.

z_o characteristic roughness parameter.

γ unit weight of water.

λ a coefficient depending upon the turbulence in flow.

ρ density of water, slugs/ft.^{3}

ρ_a density of air, slugs/ft.^{3}

τ_b shear stress on the bottom, lbs/ft.^{2}

τ_s shear stress on the water surface, lbs/ft.^{2}

Theoretical Considerations

The analysis of wind tide data involves numerous variables which include the wind velocity and the energy transfer between the air and water, which in turn depend upon the wave conditions, temperature gradients, etc. The depth and the geometrical shape of the body of water and the configuration and roughness of the bottom are other factors of importance. The stratification (layers of different density) of water may influence the results considerably, and in very large bodies of water the rotation of the earth, and the variation of atmospheric pressure to the surface elevation and directions of currents may also be important. To solve the problem theoretically, several assumptions must be made which include the flow conditions and viscosity at and near

the air-water boundary. Several investigators have studied this problem using different approaches and assumptions.

Consider the simple two-dimensional condition represented in the following sketch.

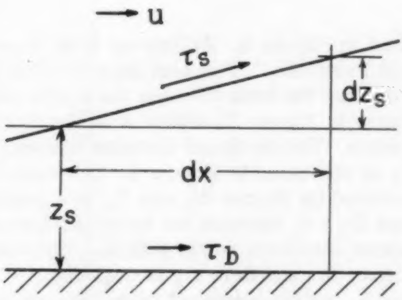

FIGURE 2

For steady conditions, the force equation for unit width perpendicular to the sketch is:

$$\frac{\gamma}{2}\left\{(z_s + dz_s)^2 - z_s{}^2\right\} = (\tau_s + \tau_b)\,dx\ . \qquad (1)$$

Disregarding terms of order higher than the first of dx and dz_s the differential equation of the water surface is

$$\frac{dz_s}{dx} = \frac{(\tau_s + \tau_b)}{\gamma z_s}\ . \qquad (2)$$

Keulegan(7) has chosen to express the bottom shear τ_b, as a function of the surface shear, τ_s, by the relation

$$\tau_b = \tau_s\,(\lambda - 1)\ . \qquad (3)$$

Substitution of equation 3 in equation 2 gives

$$\frac{dz_s}{dx} = \lambda\,\frac{\tau_s}{\gamma z_s}\ . \qquad (4)$$

Equation 4 is the form used by most investigators in studies of wind tides. The value of λ used in the integration of this equation depends on the particular flow theory that is adopted. Following is a summary of the forms to which Equation 4 has been reduced by the various investigators to compute wind tides:

1. Hellstrom(3) integrates equation 4 to get

$$z_s{}^2 = \frac{2\lambda\,\tau_s}{\gamma}\,(x + C_1)\ . \qquad (5)$$

Equation 5 indicates that the water surface is parabolic in form and may be written in coordinates ζ_s and ξ as follows:

$$\zeta_s^2 = \frac{2\lambda\tau_s}{\gamma}\ \xi\ . \tag{6}$$

Equation 6 is plotted in Figure 3. Hellstrom calls Equation 6 the "Characteristic Water Surface Parabola." The next step is to cut out a portion of the parapola in length F so that the area between the curve and the x-axis is equal to Fd (shaded area in Figure 3), where F is the length of the channel and d the stillwater depth. The so-found distance between the z- and ζ_s-axis gives the constant C_1 as indicated in Figure 3. The case where the bottom is not exposed is represented by Figure 3a, and C_1 is a positive value. Figure 3b represents the case $C_1 = 0$, wherein the water surface at the beginning of the channel has the same elevation as the bottom. The case with an exposed bottom is given in Figure 3c, and here C_1 is negative.

Finally, the set-up, h, can be computed from the equation

$$h = \sqrt{\frac{2\lambda\tau_s}{\gamma}\ (x + C_1)}\ - d\ . \tag{7}$$

The nodal-line can be computed from Equation 7 for h = 0.

When the depth is great as compared with the set-up, Hellstrom gives the formula

$$h = \frac{\lambda\tau_s}{\gamma d}\ \left(x - \frac{F}{2}\right)\ . \tag{8}$$

The nodal-line for Equation 8 is at x = F/2 and the set-up at the windward shore, x = 0, is:

$$h_{x=0} = -\frac{\lambda\tau_s F}{2\gamma d} \tag{9}$$

and at the leeward shore, x = F $\quad h_{x=F} = \dfrac{\lambda\tau_s F}{2\gamma d}\ .$ (9a)

2. Langhaar's analysis(5) is based on the momentum principle, separating the effect into statical and dynamical components. The statical tide is the tide that the wind would maintain if it persisted indefinitely; the tide due to the seiches is called the dynamical tide. He gives two different formulas for statical tides, depending upon the magnitude of the wind tides.

a) For small tides where the bottom is not exposed he gives the formula:

$$h_{x=F} = \frac{\tau_s F}{2\gamma d} \tag{10}$$

where h_x = F indicates the wind tide above the original still-water at the leeward shore. The formula is identical to Equation 9a, as presented by Hellstrom for λ = 1.

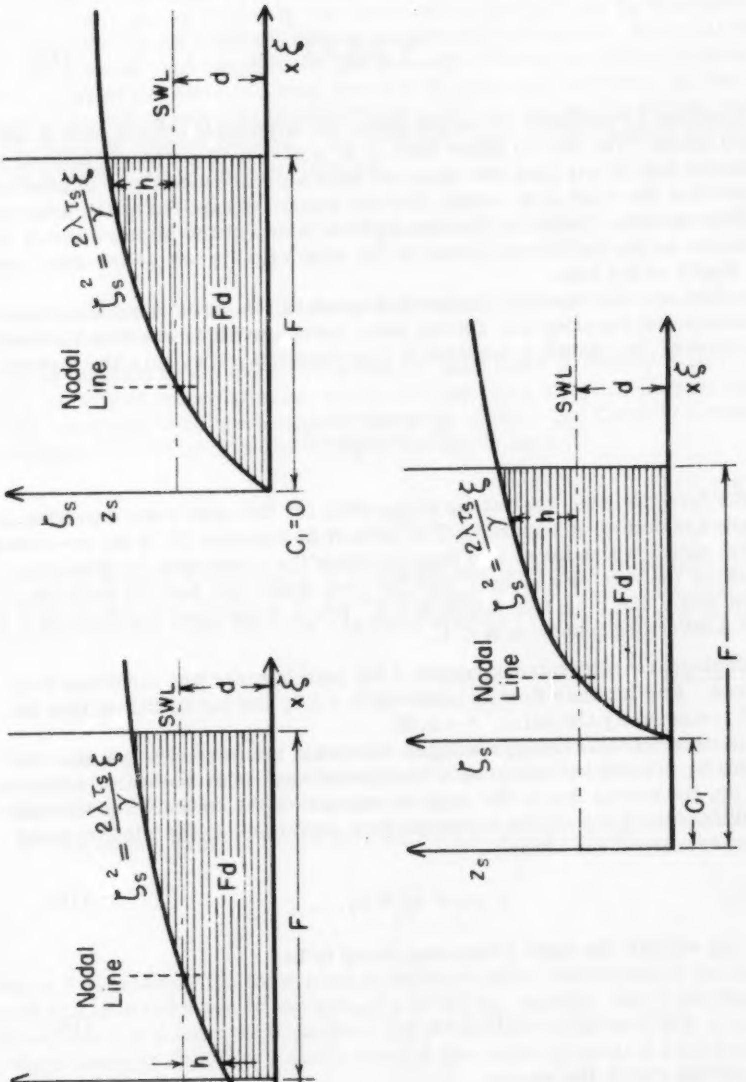
Nodal Line
SWL
Fd
F
d
h
x
ξ
z_s
ζ_s
$C_1 = 0$
C_1
$\zeta_s^2 = \frac{2\lambda\tau_s}{\gamma}\xi$

FIGURE 3

b) The case where part of the windward bottom is exposed:

$$S' = \sqrt[3]{\frac{3\,\tau_s\,F d}{\gamma}} \quad . \tag{11}$$

S' in Equation 11 indicates the depth above the horizontal bottom, and at the leeward shore. The set-up above SWL is h' = s' - d. Equation 11 determines the statical tide in any lake that does not have a pronounced taper of planform, provided that the wind is so strong that the bottom is exposed at the windward end. This equation indicates that the depth of water at the leeward end of the lake varies as the two-thirds power of the wind velocity and as the cube root of the length of the lake.

The Jacksonville District, Corps of Engineers,(11) have found numerous applications for Equation 11. During their investigation it was found necessary, however, to chэnge a constant in this equation. They give the formula as:

$$h'_{x=F'} = \sqrt[3]{\frac{3.373\,\tau_s\,F d N}{\gamma}} - d \quad . \tag{12}$$

h' in this formula gives the set-up above SWL for the case when a portion of the windward bottom is exposed. The term N in Equation 12 is the so-called planform factor which takes into consideration the converging or diverging planform of the lake or channel. For the case where the body of water has a constant width, the planform factor N = 1. For a converging planform $N > 1$, and for a diverging planform $N < 1$.

3. Keulegan(7) integrates Equation 4 for both laminar and turbulent flow conditions. For laminar flow he obtained $\lambda = 1.5$, and for turbulent flow he adopted temporarily the value, $\lambda = 1.25$.

In his experimental study, Keulegan separated the total set-up S into two parts; (a) S_1, the set-up due to skin friction between wind and water surface; and (b) S_2, the set-up due to the form resistance of the waves. S is defined as the difference between the water-surface elevations at the windward and the leeward ends of the channel.

$$S = S_1 + S_2. \tag{13}$$

The set-up without the wave action was found to be

$$S_1 = C_2 \times 10^{-6} \ \frac{U^2}{g}\frac{F}{d} \tag{14}$$

and the set-up due to the waves,

$$S_2 = C_3 \ \frac{(U - U_o)^2\,F}{g\,d} \left(\frac{d}{F}\right)^{\frac{1}{2}} \quad . \tag{15}$$

Keulegan gives further, $C_2 = 3.3 \times 10^{-6}$, and $C_3 = 2.08 \times 10^{-4}$, so the total set-up will be

$$S = \left[3.3 \times 10^{-6}\,\frac{U^2}{gd} + 2.08 \times 10^{-4}\,\frac{(U-U_o)^2}{gd} \ \left(\frac{d}{F}\right)^{1/2}\right] F \ . \tag{16}$$

This equation was established using small-scale laboratory experiments. U_o is referred to as the "formula characteristic velocity," and is approximately 1.3 times the lowest wind velocity necessary to start waves. Keulegan states that the critical wind velocity for the genesis of waves on a large body of water is about one third as great (somewhat more than 3 ft/sec.) as the corresponding values obtained in laboratory channels, and so U_o may be omitted in Equation 16 without causing a large error. The set-up formula for the large bodies of water is then given as:

$$S = 3.3 \times 10^{-6} \left[1 + 63 \left(\frac{d}{F}\right)^{1/2}\right] \frac{U^2 F}{g d} \qquad (17)$$

and it applies when the body of water approximates the shape of a rectangular channel of uniform cross-section.

4. In addition to the above formulas for wind tides in shallow water, there are a variety of other formulas which are generally identical in form but vary in the constants to be used. Among these the Zuider Zee formula should be mentioned.(12) Originally the formula was given as

$$S = \frac{U^2 F}{800\ d} \qquad (18)$$

where S is again the difference in windward and leeward water-surface elevations, U is the wind velocity in miles per hour, F is the fetch in miles, and d the depth in feet. This formula was later modified to the following form:(13)

$$h = \frac{U^2\ F}{1400\ d} \quad \cos A \qquad (19)$$

where h is the set-up in feet above the original still-water elevation at the leeward end, U is in miles per hour, F is in miles, d is in feet, and A is the angle between the wind and tidal axis.

5. The Beach Erosion Board formula(6) was presented as

$$S = \frac{k\ \lambda \rho_a U^2 F}{\rho\ g d} \quad \cos A \qquad (20)$$

Where S represents the difference in water-surface elevations at the windward and leeward sides of the lake, ρ_a is the air density, and ρ the density of water, and k is a numerical constant approximately equal to 0.003, λ is as defined above in Equation 3 and A denotes the angle between the wind direction and the fetch.

Laboratory Equipment and Procedure

Experiments were performed in a channel 1 foot wide, 60 feet long and 1.28 feet deep, as shown in Figure 4a. The channel was constructed of wood, with one side made of plate-glass for observational purposes. The wind was generated by a blower, mounted at one end of the channel, driven by an a. c.

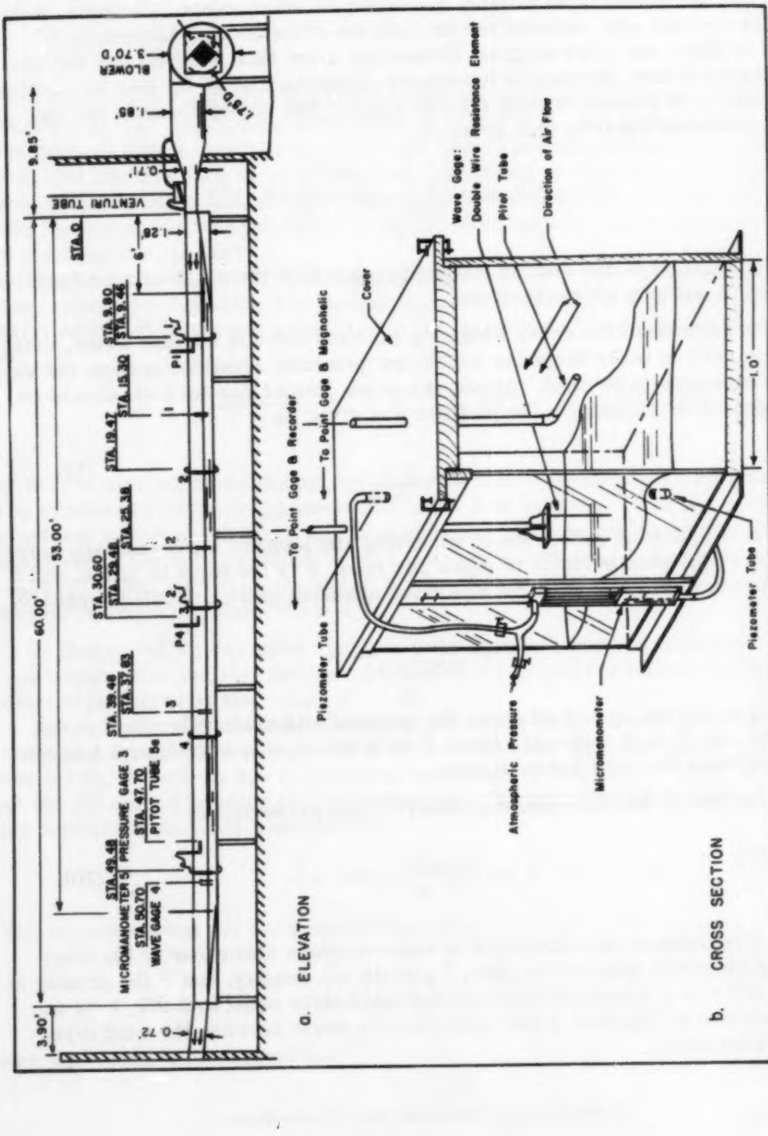

FIGURE 4 - GENERAL LABORATORY SET-UP FOR STUDY OF WIND WAVES IN SHALLOW WATER

motor. The wind velocities were varied from 0 to approximately 50 ft/sec. by adjustment of the air intake area of the blower. To straighten the wind flow upon entering the channel, a honeycomb was set between the blower and the channel. To guide the wind gradually on and off of the water surface, sloping beaches (slope approximately 1:15) were set at the beginning and the end of the channel, as shown in Figure 4a. The downwind (leeward) beach served also as a wave absorber to reduce the effect of wave reflection. The discharge of air was measured by a Venturi throat at the inlet. The Venturi throat was used to obtain approximately the desired wind velocity. The final wind velocity measurements were made, however, by using Pitot tubes so mounted to permit traversing of the channel.

Piezometer openings were installed on the top and the bottom of the channel at five locations along the centerline. The openings were connected to piezometers, as shown in Figure 4b, so that both the water depth and the total pressure, above atmospheric, could be read directly. The difference between the two readings gave the inside air pressure, and the drop in pressure between successive piezometers was used to determine corrections to be applied to the measured water-surface profiles. To check the latter measurement, three draft gages were connected to the piezometer opening on the top of the channel at the locations of micropiezometers 1, 3, and 5 as shown in Figure 4a, and the pressure readings were made simultaneously with those of the micropiezometers. These two readings always agreed very closely. Any difference indicated a faulty connection or a clogging of the piezometer opening, and corrections were made at once.

The wave heights and periods were measured at four locations, as indicated in Figure 4, by the use of parallel-wire resistance elements connected to Brush recorders.

Procedure

The desired wind velocity was obtained by adjusting the air inlet of the blower to the proper size. The blower then was shut off and the ends of the channel were closed so that no air movement could occur in the channel and influence the initial still-water elevation. When the water surface had calmed completely, the still-water elevation was determined at the location of each of the five piezometers. The blower then was started and the wind velocity profiles obtained at the three locations along the channel, as shown in Figure 4a. In later tests, only the wind velocity profile at the middle of the channel was obtained. After letting the blower run at least a half an hour, and when there was every indication that the flow condition had stabilized the average water-surface profile was measured. Because of side wall friction and the energy transfer from wind to waves, the pressure in the channel dropped gradually as the wind passed through the channel, and the measured water-surface profiles were the result of combined action of wind drag and pressure differences. To eliminate the effect of the variable pressure, the pressures were measured at each of five piezometers, as already described, and the measured surface profiles were corrected using the average pressure from these five measurements as their basis.

Experiments were performed using the following bottom roughness:

1. A smooth bottom was represented by the original bottom, painted with white oxide primer paint. The roughness for this type of bottom was determined by steady-state flow of water in a laboratory flume and was found to be k_s = 0.0135 foot in equivalent sand roughness, and the Mannings n = 0.0116.

The experiments were completed with eight different depths (0.05; 0.075; 0.100, 0.150; 0.200; 0.250; 0.300; and 0.370 ft.) each of which was combined with 5 different wind velocities (approximately 11; 15; 20; 25; and 33 ft/sec.).

2. A rough bottom condition was obtained by covering the smooth painted bottom with a 7/8 inch expanded metal lath, as shown in Figure 5. The equivalent sand roughness k_s was found to be 0.0635 foot, and the Mannings n = 0.0207. The experiments were made with 5 different depths (0.050; 0.075; 0.100; 0.200; and 0.370 ft.) each combined with the five different wind velocities mentioned above.

3. A combination of the rough bottom and cheese cloth in the channel was fastened to the bottom across the entire width of the channel. The top of each strip of cloth was made to float by the use of a thin strip of balsawood. The buoyance of the cloth was kept to a minimum so that it could easily follow the current and the motion of water particles, as do natural grasses. The height of the cloth was approximately 0.30 foot, and constant for all runs; hence, for the deepest depth of 0.37 foot used in the experiments, the top of the cloth was slightly below the SWL and for shallower depths it floated at SWL. One piece of cloth was used for each linear foot of channel. The arrangement is shown in Figure 6. The experiments for this condition were performed using four different depths (0.05; 0.10; 0.20; and 0.37 ft.) each combined with the same five wind velocities used in the previous experiments.

SUMMARY OF RESULTS

Part I - Wind Tides

A complete summary of experimental data is presented elsewhere,(14) but a few typical measured water-surface profiles are shown in Figure 7. These profiles were found to be parabolic, as was to be expected. The parabolic shape of the profiles was most pronounced for the combination of shallow water depths and high wind velocities. For the case of relatively deep water and relatively low wind velocities, the parabolic shaped water surface was found to be relatively flat, and could with sufficient accuracy be replaced by a plane (compare with assumptions for Equations 8 and 9). The theoretical water surface profiles as given by Equation 5 were compared with laboratory measurements in Figure 8 for two depths and three conditions of bottom roughness. The wind shear stress τ_s in Equation 8 was evaluated from wind velocity profiles as described below in Part II. The coefficient λ was determined experimentally as described elsewhere.(15) The constant C_1 was evaluated by the method given by Hellstrom,(3) and described by Equation 5. The agreement between the actual measurements and the theory was found to be good except for Run 70 in Figure 8e, where the measured profile was about 30 per cent steeper than the theory indicated. For cases with exposed bottom, the extent of exposure was not always predicted accurately, and it seemed that the extent of exposure was greater than the theory predicted. This may be a phenomenon which depends upon the experimental condition and such effects as surface tension, etc. In laboratory experiments it is very difficult to determine the exact location of the water line on a level bottom. Also, the surface tension may have considerable effect in the very shallow region.

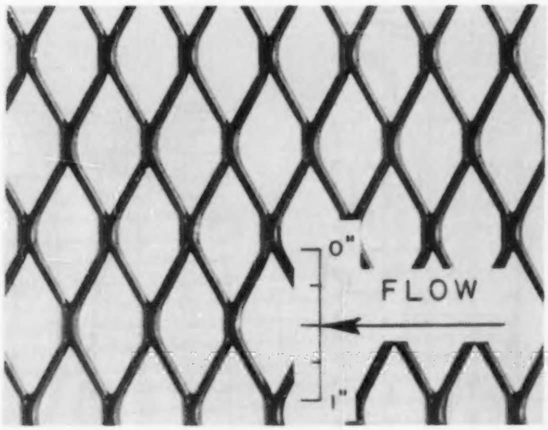

Fig. 5. Expanded metal lath used for channel bottom roughness.

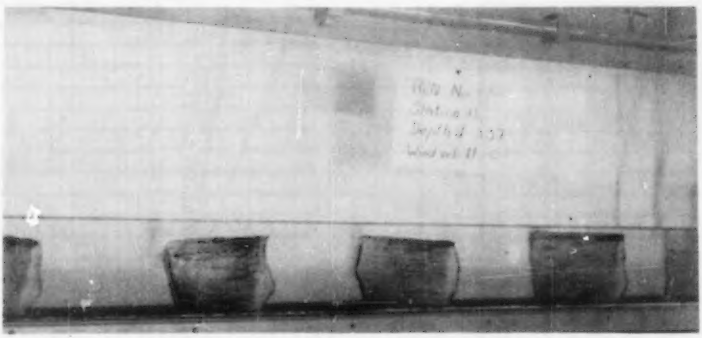

Fig. 6. Cheesecloth strips used in channel to simulate vegetation.

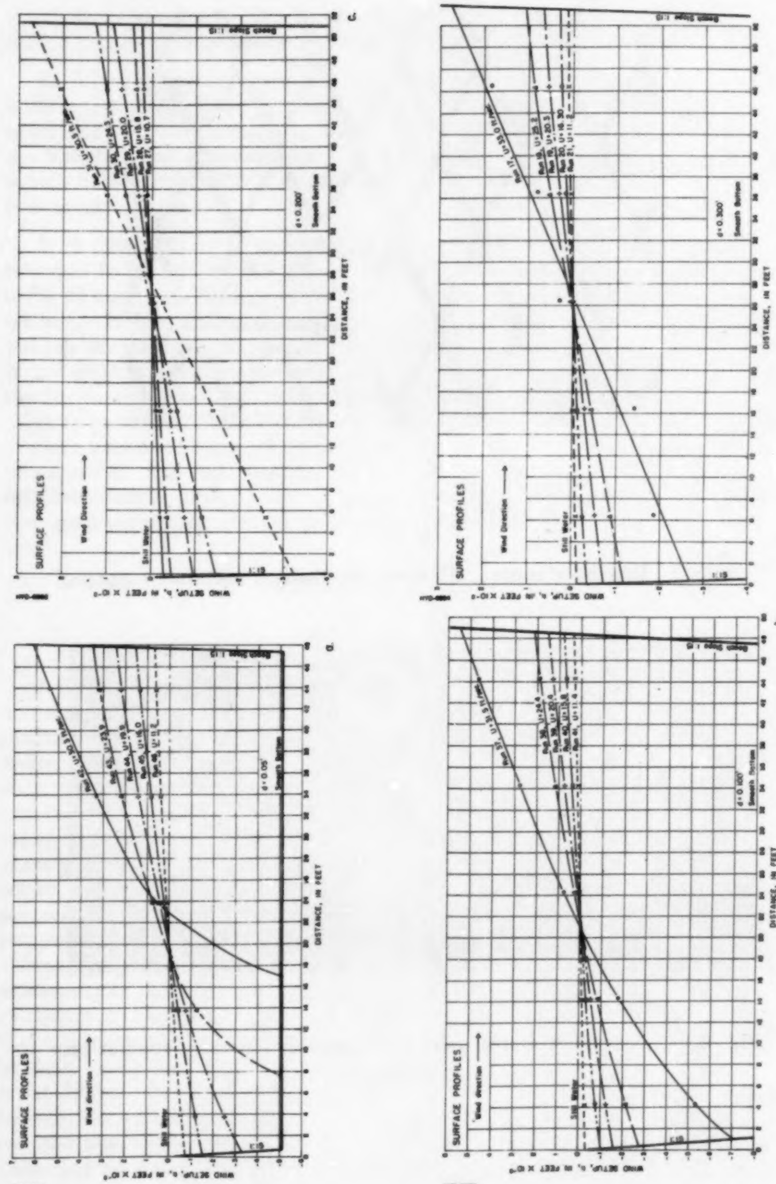

FIG. 7 - TYPICAL WATER SURFACE PROFILES FOR VARIOUS WATER DEPTHS AND WIND SPEEDS

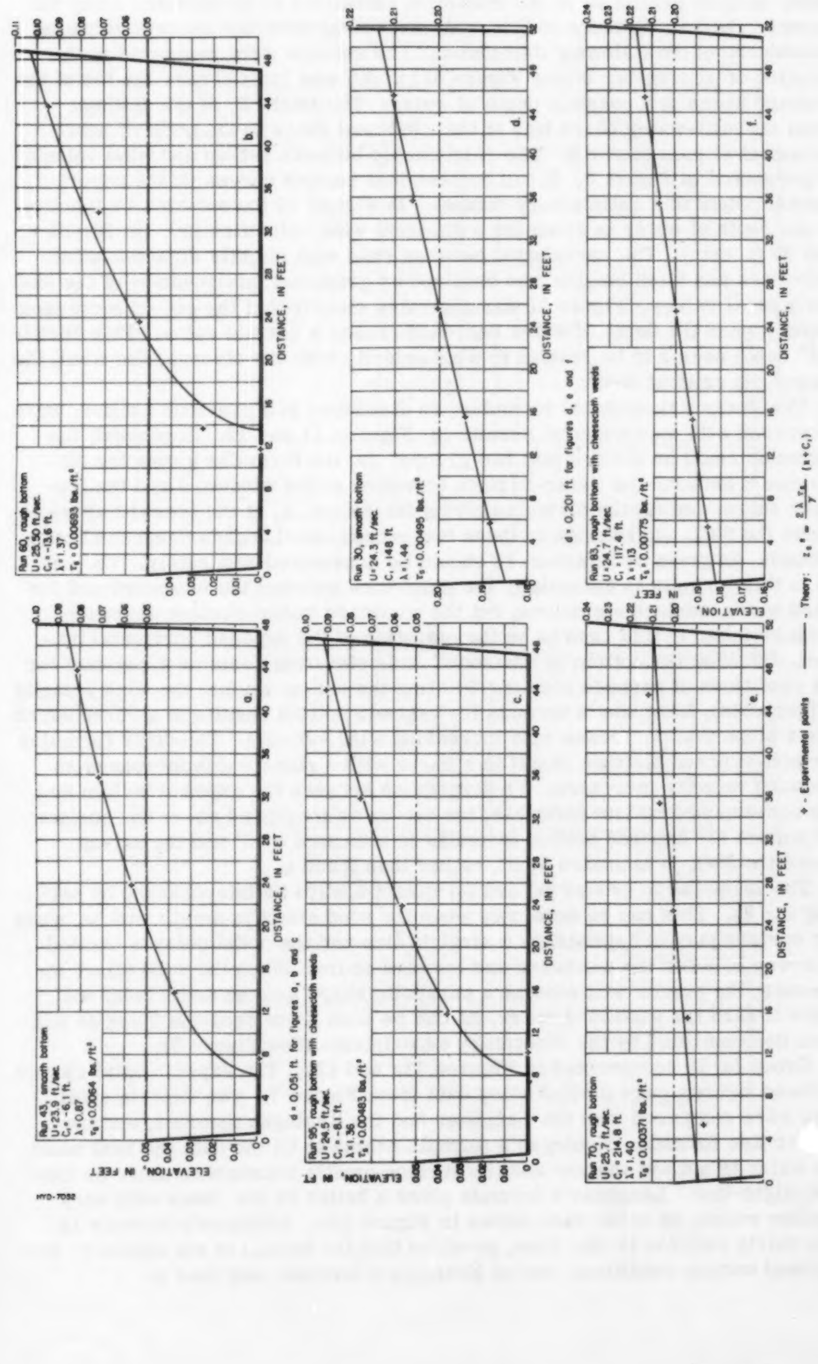

FIGURE 8 - COMPARISON OF MEASURED WATER SURFACE PROFILES WITH THEORY

In most practical cases, one is not as interested in the exact shape of the water surface profile as in the maximum elevations to be expected along the leeward shores. Because of this, only the set-up near the leeward shore is considered in the following discussion. The set-ups were measured at the location of piezometer 5 (see Figure 4a) which was located near the toe of the leeward beach in a constant depth of water. The fetch, F, is the distance from the still-water beach line at the windward shore to the point of measurement at piezometer 5. The relationship between set-up and wind velocity is presented in Figure 9. It can be seen that smooth curves fit the experimental points in a satisfactory manner. In Figure 10 the set-up as a function of the depth of water is given for 5 different wind velocities (10; 15; 20; 25; and 39 ft. sec.). The correlation between runs with slightly different wind velocities and fetch lengths was obtained by graphical interpolation of the experimental values. Figure 10 demonstrates clearly that the set-up increases rapidly when the depth of water decreases below a certain value. This "critical" depth seems to be related to wind velocity with the stronger the wind, the larger the critical depth.

The various theoretical formulas, as discussed in a previous section, were compared with experimental results in Figures 11 and 12. In general, the formulas could be divided into two groups: (a) the formulas giving the difference S between the water-surface elevation at the windward and the leeward shore, and (b) the formulas giving the set-up, h, at the leeward shore above the SWL. In addition to these two groups, the Langhaar exposed bottom formula, as given by Equation 11 should be considered separately. The set-up in this formula is essentially the difference between the windward and leeward water-surface elevations, but the windward water-surface elevation must be taken in this case to be the elevation of the exposed horizontal bottom. Because the bottom is horizontal, this elevation remains a constant for all conditions of exposed bottom. Plotting the set-up against the wind velocity (Figure 11a), Langhaar's formula for exposed bottom results in a curve which has a slope that decreases with increasing wind velocity. The other formulas for non-exposed bottoms result in a curve with a slope which increases as the wind velocity increases. To distinguish between the exposed bottom and the non-exposed bottom formulas, the set-up as measured above the horizontal bottom for exposed bottom formulas is indicated by S' and the set-up above the SWL is indicated by h', rather than S and h.

The formulas in groups (a) and (b) cannot always be interchanged by setting S = 2h. This can be done only when the wind effect is small; that is, when the surface profile approaches a straight line and the nodal point is located half-way between the windward and leeward shore. When the wind effect increases, the profile will assume a parabolic shape and the nodal point will move toward the windward shore, as can be seen from Equation 7 and as has been demonstrated by the laboratory experiments (See Figure 7).

Group (a) is represented in Figures 11a and 12a. The experimental curves in these figures were plotted using data from Figure 7. The experimental data were compared with the Langhaar and the Keulegan theories, and the Zuider Zee formula. Keulegan's formula seems to fit the data the best when the water is not too shallow and the surface profile approaches more or less a straight-line. Langhaar's formula gives a better fit for cases with very shallow water, as is the case shown in Figure 11a. Keulegan's formula is also fairly reliable in this case, provided that the bottom is not exposed. For exposed bottom conditions, use of Keulegan's formula may lead to

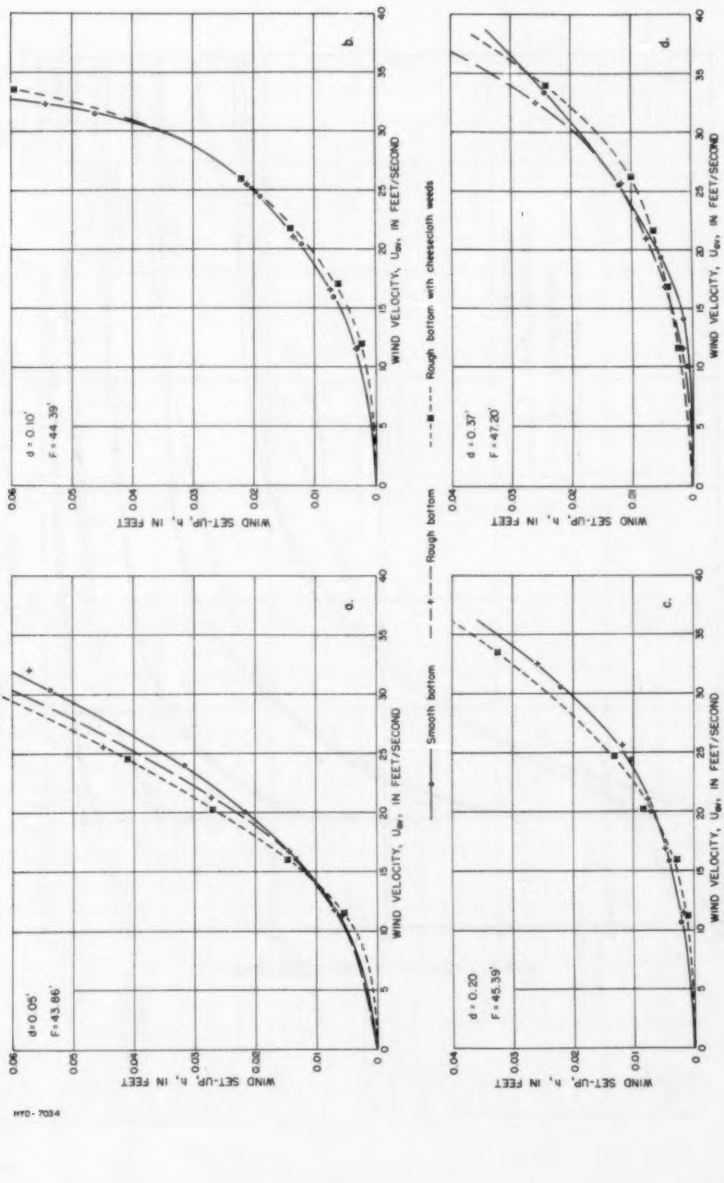

FIG. 9 - WIND SET-UP AS A FUNCTION OF WIND VELOCITY FOR VARIOUS BOTTOM ROUGHNESSES

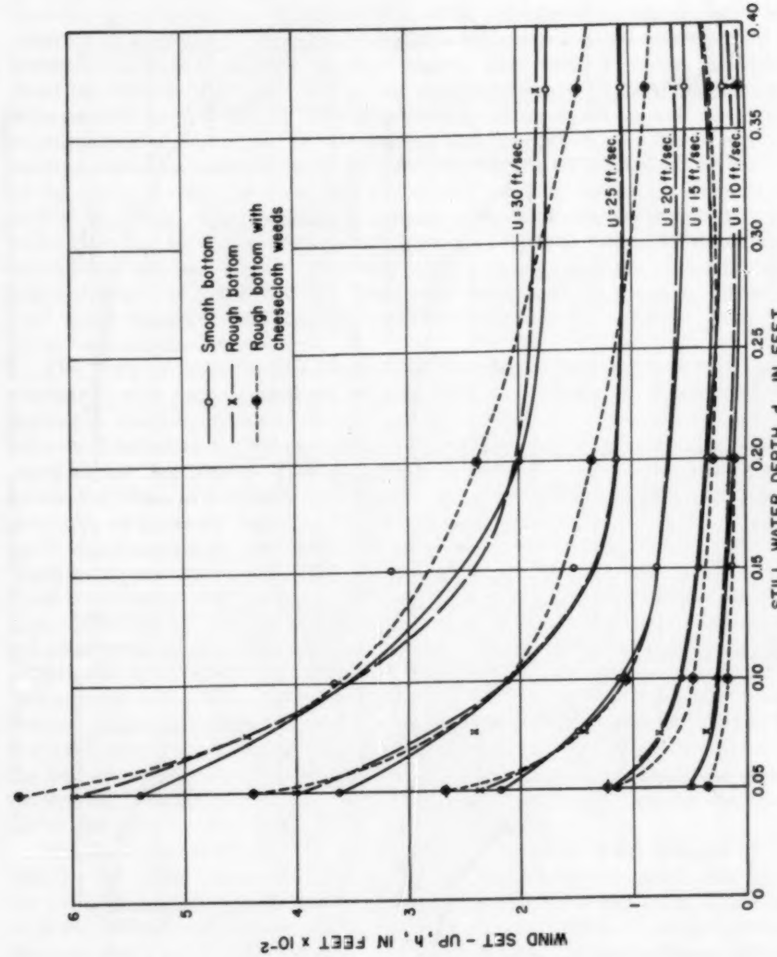

FIGURE 10 · WIND SET - UP AS A FUNCTION OF DEPTH FOR SEVERAL WIND VELOCITIES

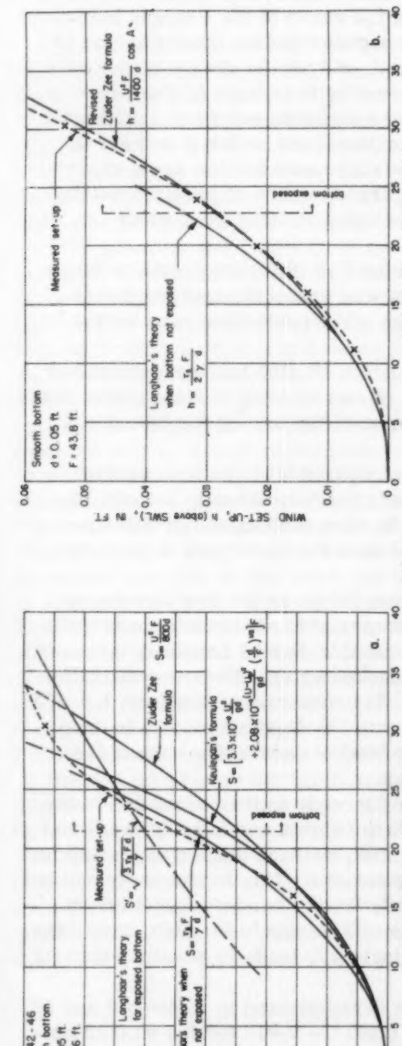

FIG. 11

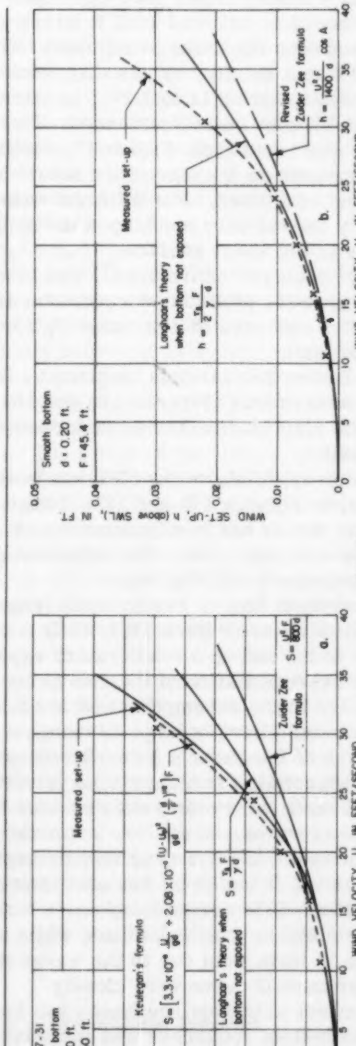

FIG. 12

COMPARISON OF VARIOUS THEORIES WITH EXPERIMENTAL LABORATORY RESULTS

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overestimation of the set-up, depending upon the degree of exposure. Langhaar's theory shows good agreement when the curve of the non-exposed bottom theory is followed until it intersects the curve of the exposed bottom theory and then the latter is followed. The exposed bottom theory seems to give somewhat smaller values than measured, so that the change of the constant 3.0 in Equation 11 to 3.373, as introduced by the Corps of Engineers in Equation 12, may be recommended. The wind shear stress as in Langhaar's formula was taken from Figure 17, which is discussed in detail in Part II, and represents an average of the actual measurements for the same experiments and conditions. For different cases, the accuracy of prediction will obviously depend very much upon the proper estimation of wind shear stresses at the water surface.

U_o, in Keulegan's formula,(7) was given for five different depths of water. These data were plotted and a smooth curve was drawn through the points. This curve was used to determine U_o for the given particular runs in this series of tests.

The Zuider Zee formula as given by Equation 18 also has been compared with measurements (Figures 11a and 12a). It can be seen in both figures that use of the formula leads to an underestimation of the set-up for laboratory experiments.

The set-up, h, above the SWL has been compared with the formulas of group (b) in Figures 11b and 12b. Langhaar's formula actually belongs to group (a), but as has been pointed out, $S = 2h$ when the surface profile approaches a straight line. The experimental data were obtained at the location of piezometer 5 (Fig. 4a).

The revised Zuider Zee formula (Equation 19) gives the best agreement for shallow water (Figure 11b), while for deeper water it leads to underestimations of the set-up for laboratory experiments. Use of Langhaar's formula leads to overestimation of the set-up for shallow water. The overestimation may be due to the assumption that $h = S/2$. In actual cases, however, $h < S/2$ when the wind effect is large as compared with the depth of water. In deep water, use of Langhaar's formula seems to lead to underestimation of the set-up, especially for higher wind velocities.

Langhaar's theory does not consider the increase in form drag due to the presence of waves. In shallow water the wave heights are usually small and so the increase in form drag may be negligible, while in deeper water and for higher waves, it has to be included in computations. This is demonstrated in Figure 12a, where use of Langhaar's formula leads to underestimations of set-up for higher wind velocities, while use of Keulegan's formula, where the increase in form drag due to the waves is included, leads to results which fit the experimental curve very closely.

The effect of bottom roughness has been demonstrated in Figures 9 and 10. When the bottom roughness was increased from $n = 0.012$ for the smooth bottom to $n = 0.021$ for the bottom covered with expanded metal lath, the set-up at the leeward end did not change, except for very shallow depths, as has been shown in Figures 9a and 10. For the shallowest still-water depth (0.05 foot) used in this set of experiments, the set-up was further increased by adding cheese cloth in the channel to simulate vegetation; then the shape of the curve indicating the set-up in Figure 10 was changed slightly. Figure 10 indicates that for deeper water depths the set-up may be smaller than for the smooth bottom, but in shallow water it seems to be definitely higher. For the depth 0.05 foot the set-up was approximately 20 per cent higher than the set-up for

the smooth bottom. The slope of the set-up curve was steeper when the cheese cloth was present than was the slope of the set-up curve for the smooth bottom.

Qualitative observations indicated that the time necessary to reach equilibrium set-up at the leeward end of the channel was longer for the rougher bottom. For the experiments with cheese cloth, it was found that the time to reach equilibrium was much longer than was the case for the experiments with the smooth bottom. On the other hand it was found that the roughness acted to dampen oscillations. For the smooth bottom the set-up usually exceeded for a short time the equilibrium position when the wind was started, and came to rest only after many oscillations about the equilibrium.

The direction and intensity of current velocities in the water can be visualized by a study of Figure 13, where strips of cheese cloth were used to simulate vegetation. Figure 13a represents the case of relatively deep water (0.37 ft.) as compared with the height of the cloth, and the top of the cloth (approximately 0.30 ft. high) floats slightly below the still-water surface (see Figure 13a). When the wind velocity was increased, the magnitude of the drift and return flow also increased. The effect of the return flow along the bottom was to incline the cloth in the direction opposite to that of the wind (marked with an arrow in the pictures). The effect of the return current was to pull the buoyant top of the cloth below the water surface. At higher wind velocities, the entire cloth was subjected to the action of the return flow. In Figure 14 the water is shallow (0.10 ft.) as compared with the height of the cloth (0.30 ft.). For lower wind velocity the cloth usually became inclined in the same direction as the wind, but at times it became inclined in the opposite direction (see Figure 14a), depending upon the buoyancy at the top of the cloth. In some cases the balsawood on the top of the cloth extended out of the water and was subjected to additional wind drag. When the wind velocity was increased above a certain magnitude, return flow was great enough to pull the top of the cloth below the water surface and out of the region of drift. Under these circumstances, the cloth reversed direction and inclined in the direction opposite to that of the wind (see Figure 14b).

Natural lake beds very often are covered with reeds and grasses which may be very dense and extend a considerable distance above the water surface. Even the wind distribution and intensity above the lake may be affected. The reeds and grass cannot follow the flow as easily as seaweed does in the ocean, and so the flow may be considerably different for this condition. At the present time there are no data available to indicate what effect these types of vegetation have on wind tides. Further study should cast more light on this important phase of the problem.

Part II - Determination of Wind Shear Stress

The wind shear stress on the water surface during the wind tide studies was calculated from the velocity distribution curves as measured by pitot tube traverses. A complete summary of the velocity distribution curves and the calculation of wind shear stress for each experimental run is presented elsewhere;(15) however, a brief discussion of the procedure is presented below.

The von Karman equation for velocity distribution in fluid flow is

$$u = \frac{2.3\, u_*}{k_0} \left(\log z - \log z_0 \right) \qquad (21)$$

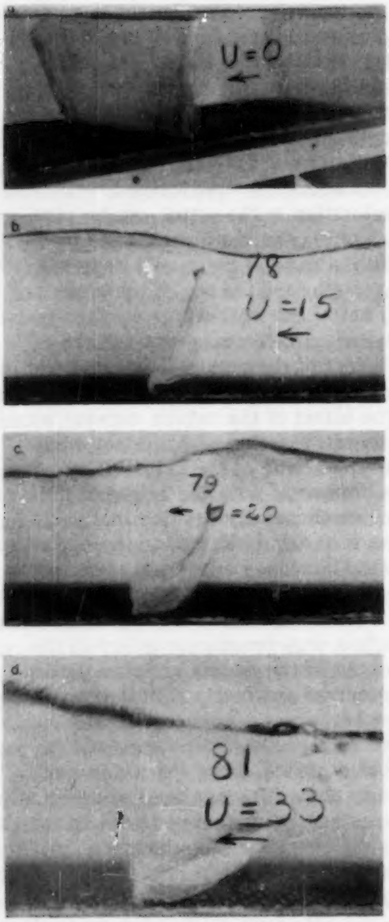

Fig. 13 - Close-up of cloth in wave action

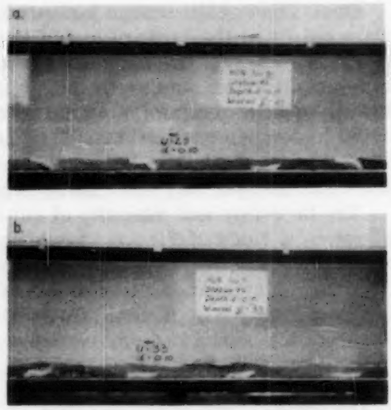

Fig. 14 - Relatively shallow water as compared with the length of cloth

where u is the velocity at elevation z, u_* is the friction velocity, i.e. $\sqrt{\tau_s/\rho_a}$, k_o is von Karman's coefficient with an assumed value of 0.40, and z_o is a characteristic roughness parameter for a given boundary. This equation can be written as

$$u = a \log z + b \tag{22}$$

where

$$a = \frac{2.3 u_*}{k_o} \quad (23) \qquad b = -\frac{2.3 u_*}{k_o} \log z_o \; . \quad (24)$$

Equation 22 appears as a straight line on semi-log graph paper; hence the velocity measurements at two different levels will be sufficient to determine the friction velocity, u_*, (hence the shear stress on the water) and the roughness parameter, z_o.

A typical set of velocity distribution curves is shown in Figure 15. These are the curves observed in connection with the wind tide data represented in Figure 7a. The values of a and b in Equation 22 were obtained as shown in Figure 16. The value of u_* then was computed from Equation 23, and the value of z_o was computed from the relationship

$$\log z_o = -\frac{b}{a} \; . \tag{25}$$

The wind shear stress, τ_s, was next computed from the relationship

$$\tau_s = \rho_a \, u_*^2 \; . \tag{26}$$

The calculated values of wind shear stress were plotted against the average wind velocity in the channel as shown in Figure 17a. In these tests the average velocity occurred at approximately 0.20 ft. above MWL. Under natural conditions (Lake Okeechobee, the Baltic, and the North Sea), the wind velocities were never measured at such a low elevation. Generally the velocities were measured at an elevation of 10 meters, which is approximately 33 ft. The wind velocities are naturally higher at the higher elevations; consequently, the relationship between shear stress and wind velocities shown in Figure 17a to field data would result in too high a value for the shear stress.

Field data from Lake Okeechobee have been made available by the Jacksonville District, Corps of Engineers. In these measurements the reference wind velocity was given for a 30-foot elevation above the crests of the significant waves. To allow comparison of these data with laboratory results, the wind velocity profiles as represented in Figure 15 for the smooth bottom condition were extended to obtain the wind velocity at 30 feet above MWL. The assumption was made that the logarithmic law applies to the higher elevations, and so the velocity would actually exist at this elevation if the channel were to be extended to an infinite height above MWL. The constants "a" and "b" in Equation 22 would not be affected by this procedure, nor would the shear stress τ_s. The only difference would be in a higher reference velocity for the given wind shear stress. The wind velocities at the 30-foot level (U_{30}) were used to replot the shear stress as a function of wind velocity in

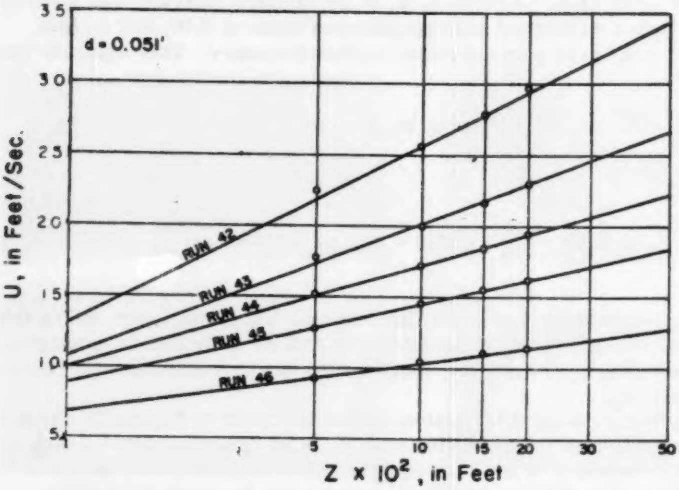

Figure 15. Variation of Wind Speed with Height Above Water Surface (Channel with Smooth Bottom)

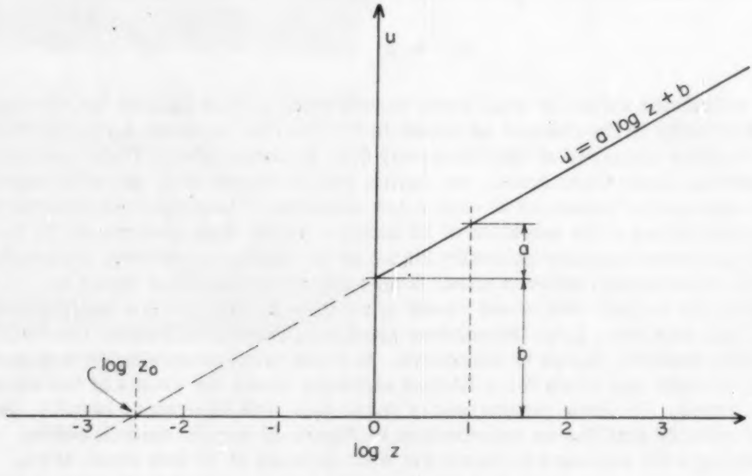

FIGURE 16

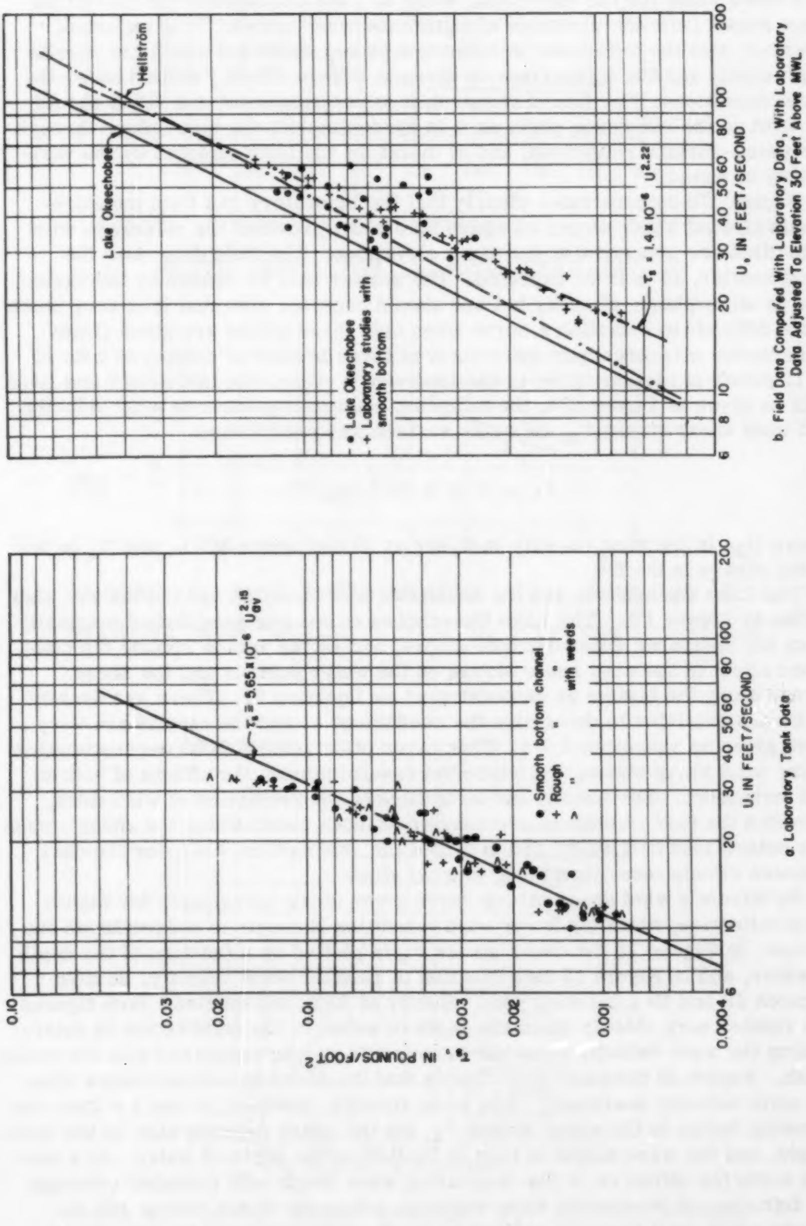

FIGURE 17- SHEAR STRESS AS A FUNCTION OF WIND VELOCITY

Figure 17b. The field data from Lake Okeechobee were plotted in the same graph. It should be noted that in the laboratory data MWL was used as the reference plane for the elevations, while at Lake Okeechobee the elevations were drawn from the elevation of significant wave crests. It is expected, however, that the difference in reference planes would not affect the results appreciably and the comparison as given in Figure 17b is justified under the given conditions. For future research it is recommended that MWL should be used as the reference plane as it is approximately the mean plane through the water-surface roughness, and is therefore the least affected by the variability of waves.

Figure 17b demonstrates clearly that the laboratory and field measurements of wind shear stress compare favorably, provided the reference wind velocities are measured at the same elevations. The field data show the most scatter, as is to be expected. The scatter may be caused by the variation in atmospheric stability but one should consider also that it is very much more difficult to establish a curve when only three points are given (Lake Okeechobee measurements were made at three levels) as compared with 10 to 12 points of measurement in the laboratory. Using the laboratory and field data as given in Figure 17b, the following relationship between wind velocity and wind shear stress τ_s, on water surface was established:

$$\tau_s = 1.4 \times 10^{-6} \, U_{30}^{2.22} \tag{27}$$

where U_{30} is the wind velocity in ft/sec at 30 feet above MWL, and τ_s is the shear stress in lbs ft^2.

The Lake Okeechobee and the Hellstrom's(3) shear stress curves are also shown in Figure 17b. The Lake Okeechobee curve was established originally from the measured water-surface slopes, and so the values include the combined effect of the wind shear stress on the water surface and the shear stress along the bottom as demonstrated by Equation 31. There are usually no means available to determine the coefficient λ, and the results are very often given by assuming $\lambda = 1$. This assumption results in an overestimation of the wind shear stress, τ_s, since the result includes the effects of bottom and turbulence. The results can be applied to the prediction of wind tides, provided the flow conditions are similar to those under which the shear stress was determined originally, and provided the assumption $\lambda = 1$, for the data includes effects other than those of wind alone.

Hellstrom's wind shear stress curve gives close agreements for higher wind velocities, while for lower wind velocities it seems to overestimate the stress. In Figure 18 the shear stress τ_s is plotted as a function of the depth of water, and in Figure 19 as a function of relative wave velocity. In both Figures 18 and 19 a constant wind velocity of 20 ft/sec existed. Both figures are related very closely since the depth of water is the main factor in determining the wave velocity when the wave length is long compared with the water depth. Figure 19 demonstrates clearly that the shear stress increases when the wave velocity decreases. The wave velocity, however, is not the lone contributing factor to the shear stress τ_s, but the latter depends also on the wave height, and the wave height in turn is limited by the depth of water. At a certain depth the influence of the decreasing wave height will probably outweigh the influence of decreasing wave velocity, and so the shear stress will decrease again until for very shallow water τ_s probably could be computed by

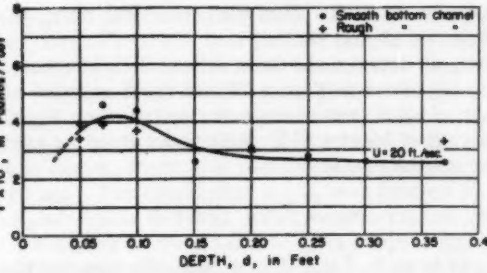

FIGURE 18 - WIND SHEAR STRESS AS A FUNCTION OF WATER DEPTH FOR U_{av} = 20 FEET/SECOND

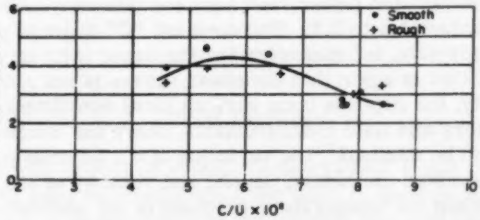

FIGURE 19 - WIND SHEAR STRESS AS A FUNCTION OF RELATIVE WAVE VELOCITY FOR U_{av} = 20 FEET / SECOND

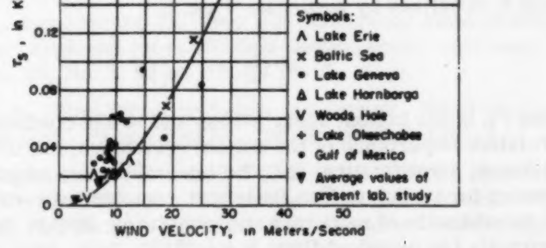

FIGURE 20 - WIND SHEAR STRESS, τ_s, AS DETERMINED BY VARIOUS INVESTIGATORS

using the smooth-plate formula. This characteristic of τ_s is demonstrated very clearly in Figures 18 and 19.

The values of τ_s as determined from different field measurements by Hellstrom[3] were replotted in Figure 20 and supplemented with the present average laboratory results (reference velocity based on U_{av}) and with measurements on the Gulf of Mexico.[16] Hellstrom gives an average relationship for the shear stress τ_s as:

$$\tau_s = 3.7 \times 10^{-4} U^{1.8} \qquad (28)$$

In Equation 28 τ_s is in kg/m^2 and U in m/sec. Comparing the various field and laboratory measurements, it is interesting to note that the empirical τ_s values, as found for different locations, may vary considerably. The results for a given location and condition, however, could be fitted usually very closely to a single empirical curve. Most of the investigators give the relationship between the wind velocity and the shear stress in the form $\tau_s = CU^m$. The values for constant "C" and power "m" vary considerably. The power "m" lies between the values 1.5 to 2.5. The constant "C" depends naturally on the units used in the formula, but may vary for the same units by a factor of ± 2. The conclusion of this is again that the shear stress is not a constant for a given wind velocity, but depends upon various local conditions as discussed above. In laboratory and field measurements where the temperature gradient may be assumed to be adiabatic, the variation of τ_s depends probably on the wave conditions, such as the height, length, and velocity of the wave and the depth of water. When the temperature gradient is not adiabatic, the shear stress also depends upon the temperature gradient.[17]

The water-surface slope, s, also can be used to determine the wind shear stress τ_s. The generally accepted equation for the water surface slope under wind action is:

$$\frac{dz_s}{dx} = \frac{\lambda \, \tau_s}{\gamma \, z_s} \; . \qquad (29)$$

$\frac{dz_s}{dx}$ is the water-surface slope and could be measured from surface profiles and the equation solved for τ_s, provided from the coefficient λ is known.

$$\lambda \, \tau_s = \gamma \, z_s \, s \; . \qquad (30)$$

The coefficient λ is defined by Keulegan[7] as:

$$\lambda = \frac{\tau_b}{\tau_s} + 1 \; . \qquad (31)$$

In this equation τ_b is the bottom shear stress and so the coefficient λ expresses the relative importance of the bottom resistance, but it depends also upon the turbulence, stratification, etc. Hellstrom[3] after adopting the Boussinesq theory for turbulent flow finds that λ varies between the values 1.15 and 1.30 for channels of moderate and very large depths. Keulegan[7] adopted temporarily for turbulent flow, $\lambda = 1.25$.

In the present experiments the wind shear stress τ_s was computed as described above by the use of wind velocity profiles, hence λ could be

evaluated from Equation 31. In Figure 21 (a to m) the wind shear stress obtained from wind velocity profiles was plotted for each bottom and depth condition separately and average curves then were drawn through the experimental points. On the same graphs the $\lambda \tau_s$ was plotted as computed from water-surface slopes. Using the average curves λ was computed again and expressed as λ_{corr}. There was not much difference between λ and λ_{corr}. The use of average curves instead of individual points eliminates only some occasional wild points. The coefficient λ_{corr} was plotted in Figures 22a and b as a function of parameter $U\,d/\nu_a$. The trend in Figure 22 is obvious with larger λ values for higher $U\,d/\nu_a$ values. It is surprising, however, to discover that the coefficient λ could be smaller than 1. Previously it was assumed that $\lambda \geq 1$. The equally surprising result is that rough bottom gives a smaller variation for λ than the smooth bottom does. The values of λ for rough bottom conditions vary between approximately 1.0 and 1.8. For the smooth bottom λ varies between approximately 0.7 (for small $\frac{Ud}{\nu_a}$) and 2.2 (for large $\frac{Ud}{\nu_a}$). The over-all average for λ was 1.27 which in turn is very close to 1.25 as adopted temporarily by Keulegan.

CONCLUSION

The result of this series of studies indicates that small scale laboratory equipment can be used to investigate wind tides in shallow water. The limiting factor appears to be the effect of the channel walls on the wind velocity distribution over the water. Whether this limitation is a serious effect should be investigated further.

ACKNOWLEDGMENTS

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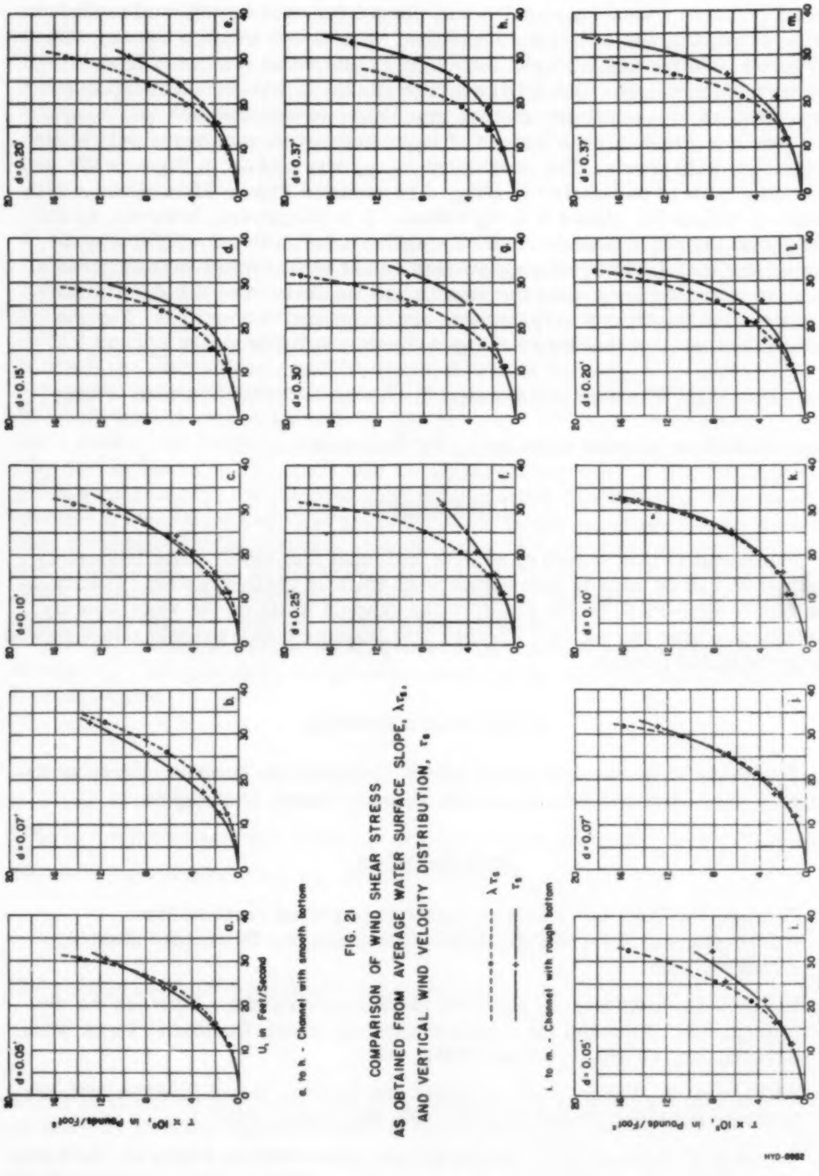

FIG. 21
COMPARISON OF WIND SHEAR STRESS
AS OBTAINED FROM AVERAGE WATER SURFACE SLOPE, $\lambda\tau_s$,
AND VERTICAL WIND VELOCITY DISTRIBUTION, τ_s

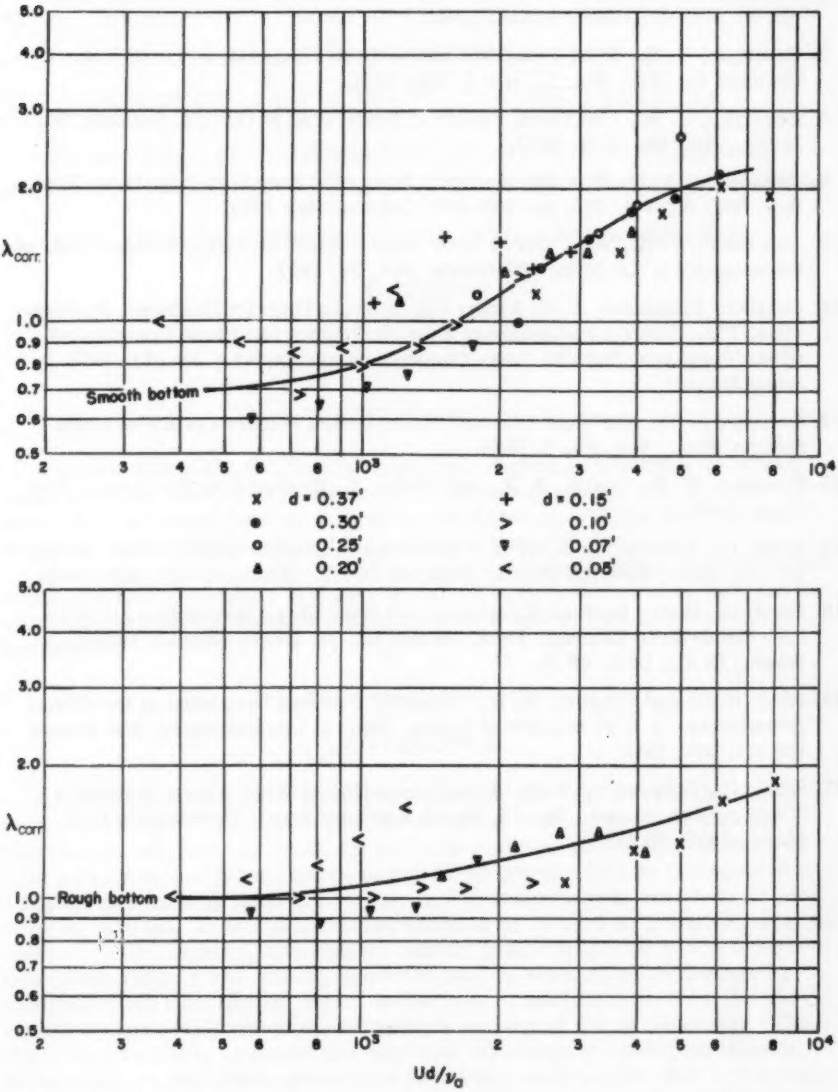

FIG. 22 - COEFFICIENT λ AS A FUNCTION OF PARAMETER Ud/ν_o

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THE LITTORAL DRIFT PROBLEM AT SHORELINE HARBORS

J. W. Johnson*
(Proc. Paper 1211)

SYNOPSIS

A harbor which fronts directly on an open shoreline and has a relatively small flow into and out of it is defined as a shoreline harbor. For such harbors where a littoral drift occurs along the shoreline certain design, construction, and maintenance problems present themselves.

This paper summarizes some of these basic considerations in generalized terms and then presents a few case histories of typical shoreline harbors for which operational information extending over a long period of years is available.

INTRODUCTION

Harbors in which sedimentation is a serious maintenance problem have been classified by Caldwell(1) into the following types: (a) river-channel harbors, (b) off-river harbors, (c) fall-line harbors, (d) channel harbors in tidal estuaries, and (e) shoreline harbors. Sediment which is transported to a harbor entrance by littoral currents is a problem only in case (e) and possibly in case (d). In this latter case, shoaling by littoral drift can occur at the entrance of the estuary, although the harbor proper might be at a remote location up the estuary and subject therefore only to sedimentation damage by material other than littoral drift. In the case of shoreline harbors the shoaling effects generally are in the immediate vicinity of the development. The discussion to follow is concerned only with the shoaling conditions at such harbors where man-made structures have been constructed. For a thorough discussion of the sedimentation problems at tidal inlets and the stabilization of such inlets for navigation purposes, the reader is referred to published works on subjects such as (a) littoral processes on sandy coasts as presented

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by Brown,[2] Mason[3] and Eaton,[4] (b) the relationship between tidal entrance and entrance area by O'Brien[5] and Robbins,[6] (c) the design and construction of jetties by Hickson and Rodolf[7] and (d) the dredging of inlets on sandy coasts by Blackman.[8]

A shoreline harbor is defined as one fronting directly on the open shore of an ocean, a bay, or a large lake and having a relatively small tidal or fresh-water flow from the tributary watershed. These harbors are generally protected from wave action by some natural feature or man-made structures (Fig. 1). Sedimentation at such harbors results primarily from the along-shore transport of sand due to wave action and littoral currents.

Littoral Processes

Wave action is the primary source of energy available at a shoreline for moving sediment. The result of waves breaking at an angle to the shoreline is to generate a longshore or littoral current. It is this current, combined with the agitating action of the breaking waves, that is the important factor in causing a movement of sand along a coast line. Perhaps the first comprehensive discussion of these processes and of shoreline development was by Gilbert.[9] Johnson[10] later formulated the concepts and nomenclature that are now generally accepted by workers in this field of study. The most comprehensive statements of present-day viewpoints on the character and solution of shoreline problems which confront the engineer have been presented by Mason[3] and Eaton.[4] Briefly summarized below are some of the general principles of littoral processes. For details, however, the reader is urged to study the papers of Mason and Eaton.[3,4]

From the geological point of view a particular segment of a coast may be advancing, retreating, or in a state of approximate equilibrium. The status of such a shore segment depends upon the material balance; that is, the rate at which littoral material is delivered to the area as compared with the rate at which it is removed. Ordinarily the changes due to natural geological processes are so slow that they are of minor importance to the engineer. Of most importance to him is the effect of a man-made littoral barrier in altering the material balance by accelerating or retarding the littoral transport of sediments. The functional planning of coastal works to provide an adequate solution of shore-control problems, therefore, is contingent on an accurate knowledge of the following factors:

1) Source and character of the beach material for the "shore segment" under study. The shore segment or "physiographic unit" is understood to be so limited that the shoreline phenomena within the area is not affected by the physical conditions in adjacent areas.

2) The manner and direction of movement of material from the source to the shore segment <u>under study</u> and from this segment to other areas.

3) The rates of supply and loss of material to and from the problem segment.

The techniques and mechanical details of determining the first two of these factors is described in a recent publication of the Beach Erosion Board.[11] With respect to item (1) the definition of the limits of the shore segment and the determination of the source and character of the littoral

Fig. 1 Typical examples of shoreline harbors

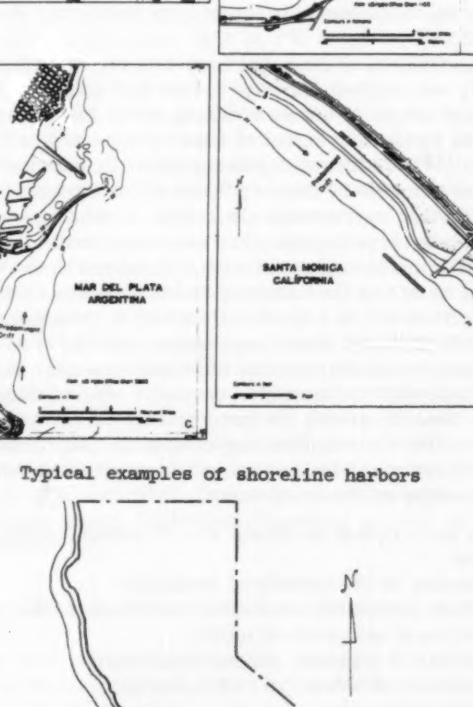

Fig. 2 - Sketch showing how the California coastline changes from a north-south to an east-west orientation at Point Conception.

material can best be supplied by studying the mechanical and mineral characteristics of the littoral materials and the general geological features of the area. In most instances it is necessary to obtain the assistance of a geologist trained in shoreline processes to determine the limits of the shore segment under study.

For example, Point Conception on the California coast was long considered to be a complete natural littoral barrier against southward drifting sand because of the rocky nature of the headland and the abrupt change in orientation of the shoreline on the downdrift side of the point (Fig. 2). It was only after extensive studies in which the heavy minerals in the sands of the adjacent beaches and streams were used as "tracers" was it established that sand was moving around this promontory.(12) This finding therefore had the effect of fixing the northern limit of the shore segment lying downcoast from Point Conception at a more northerly location than previously considered as possible.

The exact mechanism of the movement around rocky headlands has not been completely established, but it probably is a combination of sand movement by turbulent action of waves breaking at the base of the precipitous cliffs, as well as by the movement of sand by wave action in depths to as much as 80 feet.(13) Underwater photographs of sand ripples on the ocean bottom at depths from 60-80 feet, as taken at the Scripps Institute of Oceanography, indicate that appreciable movement of sand by wave action definitely occurs at relatively large depths. The amount of material moved in such depths is probably small compared with that moved in the vicinity of the shore where the result of the breaking of the waves is to create a highly turbulent flow condition that is capable of placing large amounts of sediment into suspension. Both field and model measurements(14,15) show that a relatively large percentage of sediment occurs in suspension near the plunge point of a breaker as compared with the region on either side of this point.

As stated in item (2) above, the predominant direction of littoral drift is a necessary part of the factual data required in any shoreline study. This direction of drift often can be determined from an examination of all available information by noting such evidence as,

a) accretion and erosion on either side of jetties, groins and other structures
b) shore patterns in the vicinity of headlands
c) the direction of trailing sand spits and trailing underwater bars
d) the migration of unimproved inlets
e) the movement of channels across outer bars
f) characteristics of beach and bed materials
g) current measurements

The classic work in using such evidence to establish the predominant direction of drift was that of Gulliver.(16) Johnson(10) also discussed these techniques, and more recently the Beach Erosion Board(11) has presented the details of the application of the procedures to engineering investigations.

The other factor mentioned above, and one upon which information must be available for the proper solution of shoreline problems on sandy coasts, is that of the rate of littoral transport. Unfortunately, no general relationship is available for calculating this factor from easily determined variables; although Caldwell(17) has made a start in this direction. Observations have shown that the rate of transport is a function of the sediment characteristics,

and the height, period, and direction of the waves. By use of model studies under controlled conditions, it has been shown that for a given sand size and energy content of the waves the maximum rate of littoral transport occurs when the wave crests in deep water are at an angle of about 30° with the shoreline and the waves have a steepness* of about 0.025.(18) Because of the lack of a general formula relating rate of transport to the sediment and wave characteristics, present practice in shoreline problems must rely on rates of transport which have been determined from measurements of the accretion or scour occurring over a period of time at man-made littoral barriers. A summary of all the known measured rates of drift is presented in Table I. Also given in this table is the method of determination of the rate (whether by accretion, scour, or dredging records) and the years of record for which the computations were made. The longer the record, the more reliable of course is the average rate of littoral transport. For convenience the data in Table I are grouped by geographical areas; that is, the Atlantic, Pacific, Gulf, and Great Lakes coasts of the United States and the few localities outside the United States for which information has been published.

As is to be expected, the range of rates of transport presented in Table I is considerable. The primary causes for this variation are the large ranges in values of the wave characteristics and the shoreline orientation with respect to the prevailing wave direction that occurs at the various localities; that is, the wave energy and the angle between the wave direction and the shoreline vary greatly along the coastlines of the world. Unfortunately, no wave data were obtained simultaneously with the littoral drift measurements at the various localities to permit the formulation of even an approximate relationship between the rate of littoral transport and the wave and sediment characteristics. Observations on measured rates of drift, as given in Table I, are the only reliable data now available to the engineer engaged in the design, construction, and maintenance of coastal works where the littoral movement of sediment presents a problem. The various data on drift in the United States, shown in Table I, are summarized in Figure 3, wherein the arrows indicate the direction of drift, as compiled primarily from reports of the Corps of Engineers. The indicated directions of drift are predominant directions but, as mentioned above, important reversals in drift do occur at many of the localities listed.

Littoral Drift at Shoreline Harbors

There are three basic types of man-made works at shoreline harbors which function as littoral barriers. These consists of (a) a dredged channel, (b) a jetty or groin, and (c) an offshore or detached breakwater. The littoral processes in the vicinity of such coastal works are summarized briefly as follows (4).

a) Dredged channels. Harbors are often connected with deep water offshore by means of a dredged channel through the littoral zone (Figure 4). Such a channel creates greater than normal depths with the result that littoral material accumulates therein. Sediment of small enough size to be moved in the deeper depths seaward from the end of the dredged channel, would, of course, not be affected. Both model and prototype measurements indicate that

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* Wave steepness is defined as the ratio of the wave height to wave length.

TABLE I--SUMMARY OF MEASURED RATES OF LITTORAL DRIFT

Location	Predominant Direction of Drift	Rate of Drift (cu. yds. per year)	Method of Meas. of Rate of Drift	Years of Record	Reference Number
ATLANTIC COAST (U. S.)					
Suffolk Co., N. Y.	W	200,000	Accretion	---	52
Sandy Hook, N. J.	N	493,000	"	1885-1933	59
Sandy Hook, N. J.	N	436,000	"	1933-1951	59
Asbury Park, N. J.	N	200,000	"	1922-1925	59,60
Shark River, N. J.	N	300,000	"	1947-1953	59,60
Manasquan, N. J.	N	360,000	"	1930-1931	59,60
Barnegat Inlet, N. J.	S	250,000	"	1939-1941	59,60
Absecon Inlet, N. J.	S	400,000	Erosion	1935-1946	59
Ocean City, N. J.	S	400,000	"	1935-1946	45
Cold Spring Inlet, N. J.	S	200,000	Accretion	---	46
Ocean City, Md.	S	150,000	"	1934-1936	55
Atlantic Beach, N. C.	E	29,500	"	1850-1908	41
Hillsboro Inlet, Fla.	S	75,000	"	---	63
Palm Beach, Fla.	S	150,000 to 225,000	"	1925-1939	53
GULF OF MEXICO (U.S.)					
Pinellas Co., Fla.	S	50,000	Accretion	1922-1950	51
Perdido Pass, Ala.	W	200,000	"	1934-1953	61
Galveston, Texas	E	437,500	"	1919-1934	47
PACIFIC COAST (U.S.)					
Santa Barbara, Calif.	E	280,000	Accretion	1932-1951	18,33
Oxnard Plainshore, Calif.	S	1,000,000	"	1938-1948	43
Port Hueneme, Calif.	S	500,000	"	1938-1948	50
Santa Monica, Calif.	S	270,000	"	1936-1940	54
El Segundo, Calif.	S	162,000	"	1936-1940	54
Redondo Beach, Calif.	S	30,000	"	---	54
Anaheim Bay, Calif.	E	150,000	Erosion	1937-1948	49,56
Camp Pendleton, Calif.	S	100,000	Accretion	1950-1952	58
GREAT LAKES (U.S.)					
Milwaukee Co., Wis.	S	8,000	Accretion	1894-1912	40
Racine Co., Wis.	S	40,000	"	1912-1949	44
Kenosha, Wis.	S	15,000	"	1872-1909	57
Ill. State Line to Waukegan, Ill.	S	90,000	"	---	42
Waukegan to Evanston, Ill.	S	57,000	"	---	42
South of Evanston, Ill.	S	40,000	"	---	42
OUTSIDE OF THE UNITED STATES					
Waikiki Beach, T. H.	---	10,000	Suspended load samples	--	48
Monrovia, Liberia	N	550,000	Accretion	1946-1954	64
Port Said, Egypt	E	910,000	Dredging	---	37
Port Elizabeth, S. Africa	N	600,000	Accretion	---	38
Durban, S. Africa	N	383,000	Dredging	1897-1904	39
Madras, India	N	740,000	Accretion	1886-1919	20,22
Mucuripe, Brazil	N	427,000	Accretion	1946-1950	-

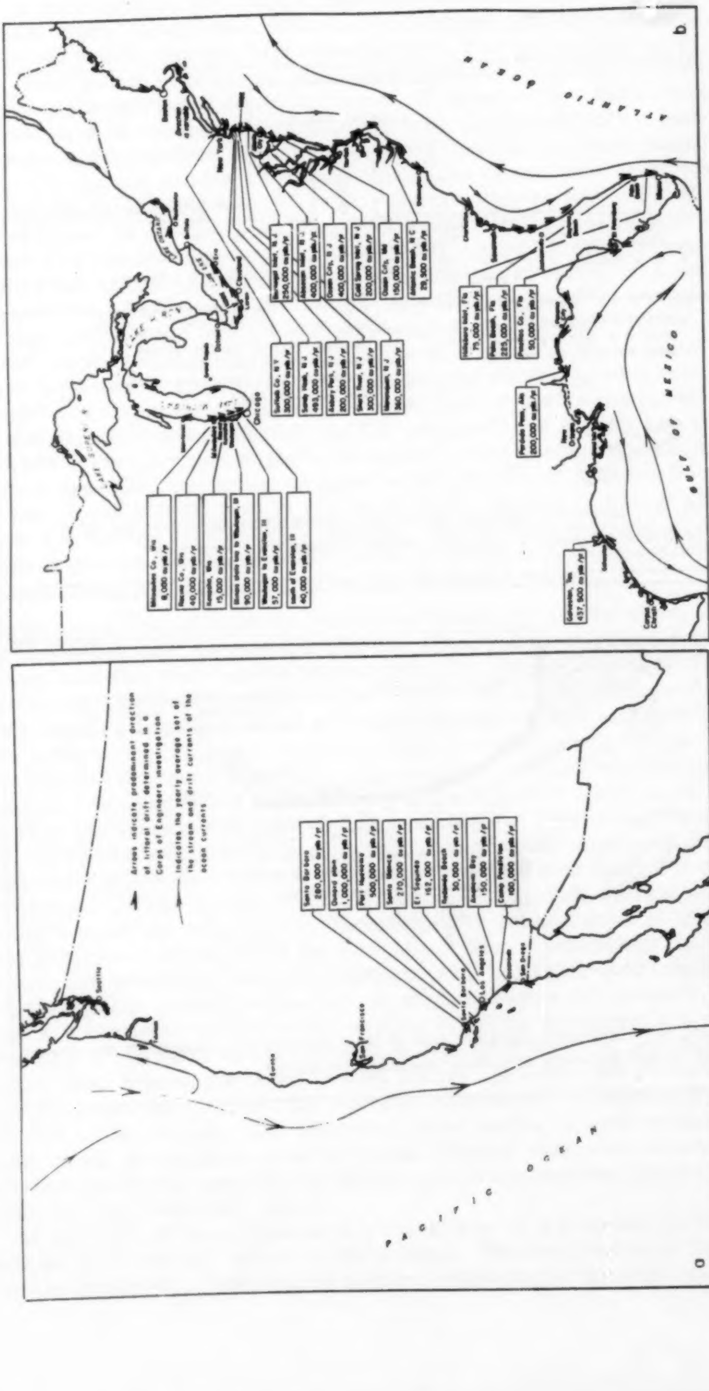


Fig. 3 Summary of observations on rates and predominant direction of littoral drift in the United States

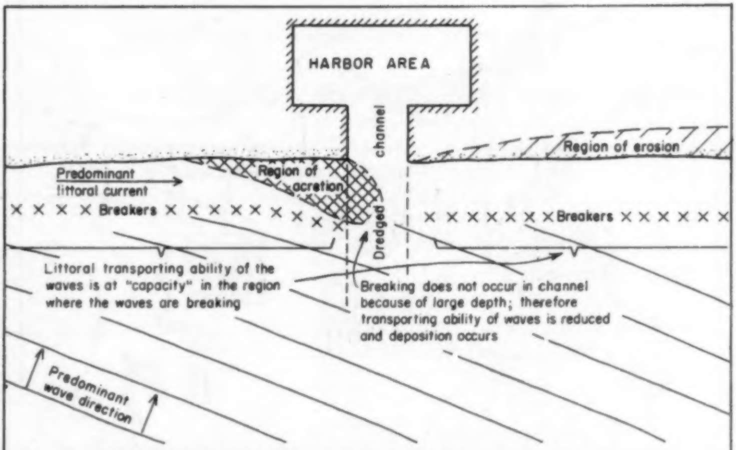


Fig. 4 - Schematic representation of the transportation, deposition and scour of littoral sediments at a channel dredged through the littoral zone.

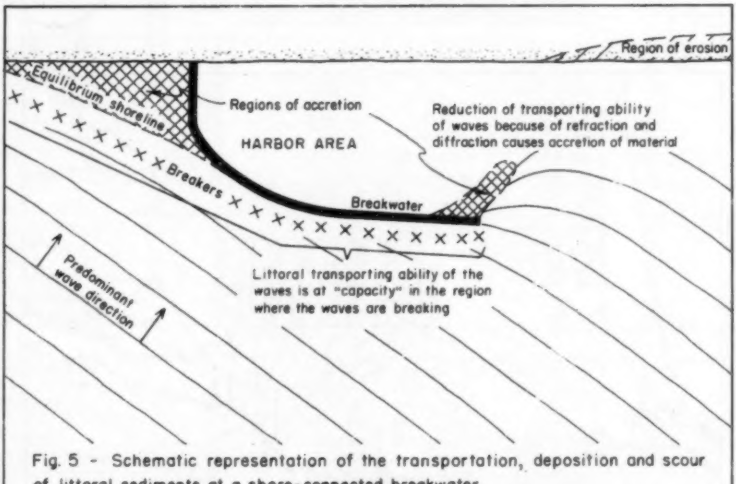


Fig. 5 - Schematic representation of the transportation, deposition and scour of littoral sediments at a shore-connected breakwater.

most of the longshore transport of material occurs in the vicinity of the breakers where the available wave energy is converted suddenly from an oscillatory motion into the form of turbulence. For that portion of the wave crest which moves over a dredged channel, however, breaking does not occur because of the increased depth, and the wave energy passes the former point of breaking to be spread by refraction and then dissipated further inshore. The degree of turbulence, therefore, is insufficient to transport material across the channel and the material accumulates as indicated in Figure 4. To maintain the channel in a navigable condition, this accumulation of littoral material must be dredged periodically. If this material is removed and redeposited on the downcoast side of the channel, normal littoral transport will occur in that region and the downcoast shoreline will remain in an equilibrium position. If, however, the channel deposits are placed elsewhere, then the supply of material to the downcoast beach is reduced and erosion and retreat of the shoreline probably will result. In a harbor such as shown in Figure 4 the action of the waves is to restore the natural littoral transport of material and thus reduce the area of the entrance to a size compatible with the tidal prism. The equilibrium size of entrance to be expected on the west coast of the United States might be estimated by the relationships between entrance area and tidal prism as given by O'Brien(5) and Robbins.(6)

If the rate of littoral drift is relatively large, the type of harbor shown in Figure 4 is seldom economically feasible. The dredged entrance is often further improved and stabilized by the construction of a shore-connected jetty or breakwater. The harbor so formed is discussed below.

b) <u>Harbors created by shore-connected breakwaters.</u> The effect of a structure which extends seaward from the shore and across the littoral zone is to act as a dam and trap the littoral drift. The impounding capacity of such a barrier is dependent upon the height of the structure, the bottom slope, and the equilibrium alignment of the shore in that region. The equilibrium alignment is one which is normal to the resultant littoral forces. Thus in Figure 5 if the original shoreline was stable with respect to the material balance and a breakwater is constructed as shown, accretion will first occur in the form of a fillet on the upcoast side with an alignment tending toward equilibrium. This will create a deficiency in material supplied to the downcoast shoreline, where erosion probably will occur with the shoreline also tending toward equilibrium. As the upcoast fillet approaches equilibrium, littoral material will move along the outer face of the breakwater and be deposited in the relatively calm water in the lee of the structure. Thus the turbulent character of the wave action upcoast from the breakwater tip is sufficient to transport littoral material at a certain capacity. At the breakwater tip, however, the waves are refracted and diffracted into the lee of the structure, and sufficient turbulence of the water is not available to transport material and deposition occurs. This deposit continues to grow toward the downcoast shoreline, and when it reaches the shoreline the material balance will be reestablished on each side of the barrier. The alignment of the harbor deposit depends primarily on the predominant wave direction. Typical examples of harbor deposits are presented below in the discussions of the harbors at Santa Barbara, California, and Fortaleza, Brazil.

The selection of the alignment of a breakwater at a proposed harbor depends on the functional aspects of the project. The termination of the outer tip of the breakwater, however, is dictated primarily by the stipulation that

the desired anchorage area is provided and that the deposit of littoral drift will interfere in the least possible way with the efficient operation of the harbor. A proper understanding of the littoral regimen of the area is the most important factor in the design procedure. Experience obtained by observations of littoral processes at nearby littoral barriers is of special value. Such observations often permit the determination of whether or not important reversals in the direction of drift occur. A study of the filling pattern of deposits in the vicinity of barriers is also of value in estimating what pattern might be expected at a proposed nearby barrier. Some typical examples of littoral drift patterns at single-breakwater harbors are presented below in the discussion of existing harbors. Also of interest are the model studies of Saint Marc and Vincent(19) on the accretion pattern at the tip of a breakwater.

In order for a harbor to be maintained in an operating condition, the sand deposit at the breakwater must be periodically removed by dredging. This dredged material should be placed on the downcoast beach to maintain a material balance thereon and reduce the recession of the shoreline resulting from erosion. For this operation to be successful the material must be placed on the downcoast beach in a location which is fully exposed to littoral forces and in depths not exceeding the limiting effective depth of the barrier. The frequency of dredging depends upon the impounding capacity of the barrier and the rate of littoral drift. That is, for the typical condition shown in Figure 6, the stockpiled material of volume V, must be removed by dredging when the distance D has decreased to the extent that ship operations into the harbor are seriously restricted. The frequency of dredging is approximately equal to the maximum allowable volume of stockpiled material divided by the average rate of littoral drift, Q_d.

For example, at Santa Barbara harbor, where the rate of drift as shown in Table I is approximately 280,000 cubic yards per year and the accretion pattern is as shown in Figure 20, dredging must be practiced every two to three years to prevent the usefulness of the harbor from being seriously impaired.

Harbors of the type shown in Figure 5, which consist of a single breakwater, are feasible when protection is required from storm waves arriving principally from only one direction. In such cases reversals of littoral drift are non-existent or unimportant. The bypassing of littoral drift past the harbor entrance may be accomplished by: (a) removing the material at the rate of accumulation, or (b) allowing the material to accumulate in a "stockpile" and removing it periodically. The method that is used at a particular locality depends primarily upon the type of dredge that is available locally. Removal at the rate of accumulation by a fixed pumping plant or a hopper dredge might be used, whereas a large capacity pipeline dredge has been the economical solution at some harbors for removing deposits which occurred over periods of two or three years.

A variation in the shape of a harbor formed by a shore-connected breakwater is the case where two breakwaters must be provided to insure protection from storm waves which may approach the entrance from various directions (Figure 1c). Pronounced reversals in the direction of littoral drift usually in such instances. Exclusion of both wave action and littoral drift from such harbors present considerations which are fundamentally opposed to each other; that is, a relatively narrow entrance between the breakwater tips is desirable to reduce wave action within the harbor, whereas a wide entrance permits considerable flexibility in the operation of removing

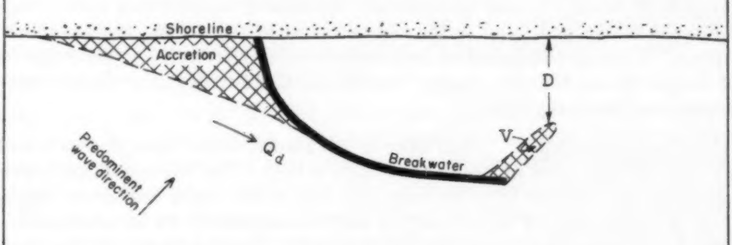


Figure 6

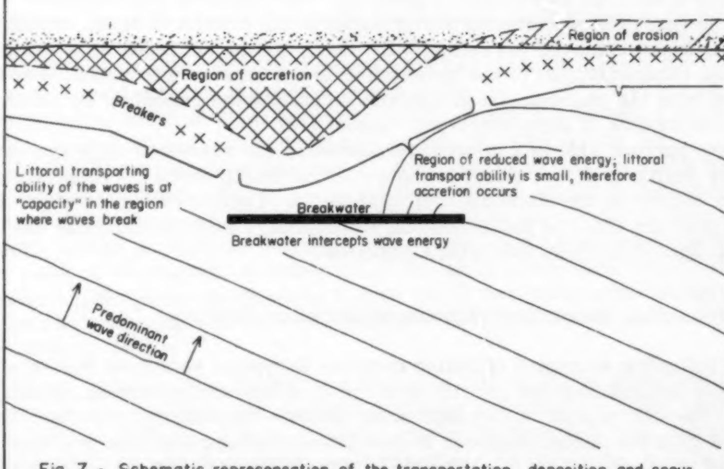


Fig. 7 - Schematic represensation of the transportation, deposition and scour of littoral sediments at a detached breakwater.

littoral drift deposits once equilibrium conditions at the adjacent shorelines have been reached. If the rate of littoral drift is relatively high and the entrance must be relatively narrow to provide adequate protection against wave action in the harbor, two solutions are possible. The first is that almost continuous bypassing of the sand by a fixed plant or hopper dredge be practiced, or second, the littoral drift be stockpiled by an auxiliary structure, as proposed for Port Hueneme, California,(50) and then periodically pumped to the downcoast side of the entrance (see Figure 23). If appreciable reversals in littoral drift occur, it may necessitate bypassing material in one direction at one season of the year and in the opposite direction during the remainder of the year. Typical examples of twin-breakwater harbors are discussed below for Salina Cruz, Mexico; Camp Pendleton, California; Port Hueneme, California; and Madras, India.

c) Detached breakwater. This type of structure intercepts the waves and creates a protected area of relatively calm water. The theory of such breakwater location is that the littoral material will move along the coast uninterrupted by the presence of the structure and consequently no maintenance problems from sediment deposition is involved. This assumption, however, is in error because the result of the refraction and diffraction of the waves behind the structure is to reduce the energy available for littoral transport in the lee of the structure as compared with the energy available on both the upcoast and downcoast shorelines (Figure 7). The result of this reduced energy is that littoral material accumulates in the protected area. If this material is not removed periodically by dredging, the accretion may eventually extend completely out to the breakwater in the form of a tombolo. For a discussion of the phenomenon of tombolo formation the reader is referred to the model studies of Saint Marc and Vincent.(19)

On the upcoast side of a detached breakwater the accretion advances beyond the region of direct effect of the structure itself, and a corresponding erosion occurs on the downcast side (Figure 7). Typical examples of harbors of this type are those at Santa Monica, California, Ceara, Brazil, and the original harbor at Santa Barbara, California.

Accretion Patterns at Shoreline Harbors

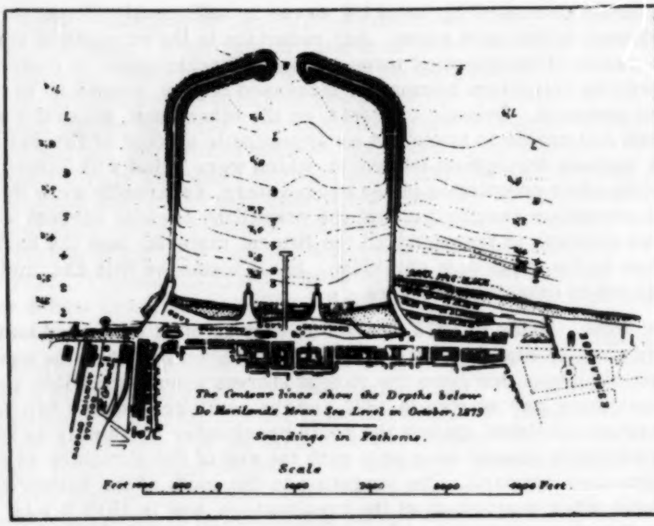
The following examples of filling patterns at typical shoreline harbors are presented to illustrate the general principles of littoral processes discussed above. The information is perhaps most valuable because the observed filling patterns give the design engineer a conception of what conditions might be expected at a harbor in a locality with a rate of littoral drift and direction of wave attack similar to the cases presented.

In this respect attention should be directed to the harbors at Madras, India, and Ceara, Brazil, which were discussed extensively in the engineering journals of the 19th century following construction of the breakwaters. The bitter lessons learned at these harbors stand as classic examples of the seriousness of the littoral drift problem. Unfortunately, these lessons were either overlooked or ignored as not being applicable to other coastlines when the harbors at Salina Cruz, Mexico; Santa Barbara, California; Santa Monica, California, etc., were constructed. Many of the mistakes made in the past apparently resulted from confusion as to the relative strength of the littoral current and the general oceanic currents existing in the particular localities. Generally, only

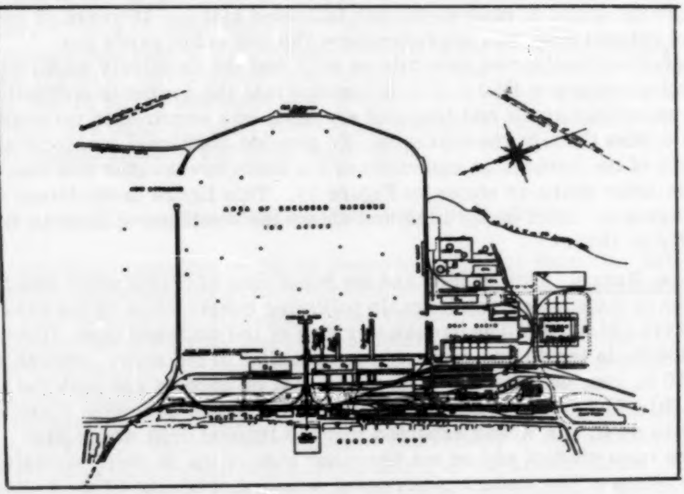
the littoral current generated by breaking waves is sufficiently strong to transport sediment of the sand sizes. Any reduction in the strength of this current, as a result of intercepting wave energy by breakwaters or distributing wave energy by refraction because of increased depths, causes an accretion of littoral material. Oceanic currents, on the other hand, generally are relatively weak and unable to transport an appreciable amount of littoral material. Many harbors throughout the world, which were filled with littoral material shortly after completion of the breakwaters, apparently were designed on the erroneous assumption that the prevailing oceanic current would be of sufficient strength to transport all the littoral material past the harbor entrance and on to the downcoast shoreline. In all instances this assumption has invariably led to disastrous results.

a) Madras, India. Prior to 1875, there was only an open roadstead on the exposed sandy coast at Madras. The breakwaters shown in Figure 8a were planned to provide protection from the violent storms which swept this coast. The north breakwater was started in 1875 and the south breakwater two years later.[20] Sand accumulated against the south breakwater so rapidly as to cause the foreshore to almost keep pace with the end of the structure as construction progressed seaward. The accretion to the south of the harbor continued to enlarge after completion of the breakwaters, and in 1910 it was necessary to close the original entrance which was 550 feet in width, and provide a new entrance which faced north (Figure 8b). Simultaneously with the formation of the accretion to the south, there was a corresponding erosion of the shoreline to the north. The extent of the accretion and erosion by 1912 is shown in Figure 9.[21] By 1919 the accretion had increased along the outer edge of the breakwater to the extent shown in Figure 10.[22] Surveys of the accretion to the south of Madras harbor indicated that for 33 years of record the rate of littoral drift was approximately 750,000 cubic yards per year.[21,22,23] Considering this rate of drift and the relatively small width of the original entrance (550 ft.), it is obvious that the available stockpile area was relatively small and frequent dredging was required to maintain navigable depths through the entrance. To provide additional stockpile area to the south of the harbor, an extension of the south breakwater was constructed in later years as shown in Figure 11. This figure is the latest available hydrographic chart of the area and shows the condition of Madras Harbor and vicinity in 1947.

b) Ceara, Brazil. This harbor and its rapid rate of filling were discussed extensively in the engineering journals following construction of the breakwater in 1875.[24,25,26] The breakwater was of the detached type, approximately 1400 ft. in length and more or less parallel to the shore, with an iron viaduct 750 ft. long designed to connect the east or upcoast end with the shore (Figure 12b). The prevailing drift in this section of the Brazilian coast is from east to west, and it was expected that the littoral drift would pass through the open viaduct and on out the other side of the harbor. Actually, as the construction of the viaduct progressed seaward, a tongue of sand also moved seaward, reached the detached breakwater, and completely closed the passage through the viaduct.[26] Sand eventually moved around the end of the breakwater and formed a bar which joined the downcoast shoreline. An enclosed pool of water was formed between this sand bar, the breakwater, and the accretion at the viaduct. Attempts to maintain the harbor in an operational condition by dredging apparently were made. The navigation chart shown in



(a) Madras Harbor in 1879 (after Vernon-Harcourt).



(b) Madras Harbor in 1911 (after Cunningham).

Fig. 8

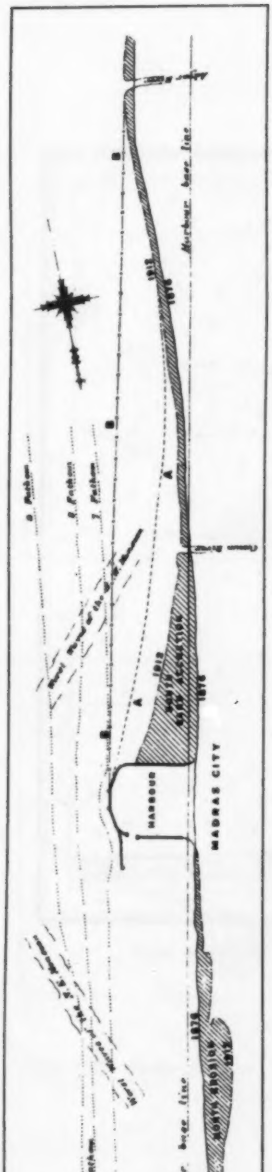


Fig. 9 Accretion and Scour at Madras Harbor in 1912 (Min. Proc. Inst. C. E., 1912-13)

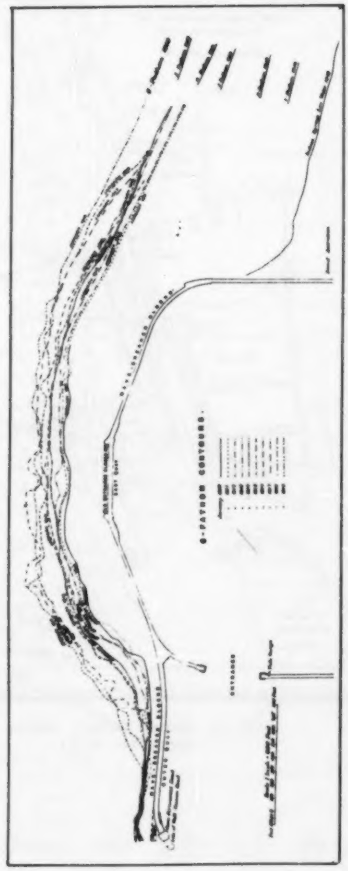


Fig. 10 Accretion along outer portion of Madras Harbor breakwater in 1919 (Min. Proc. Inst. C. E., 1919-1920).

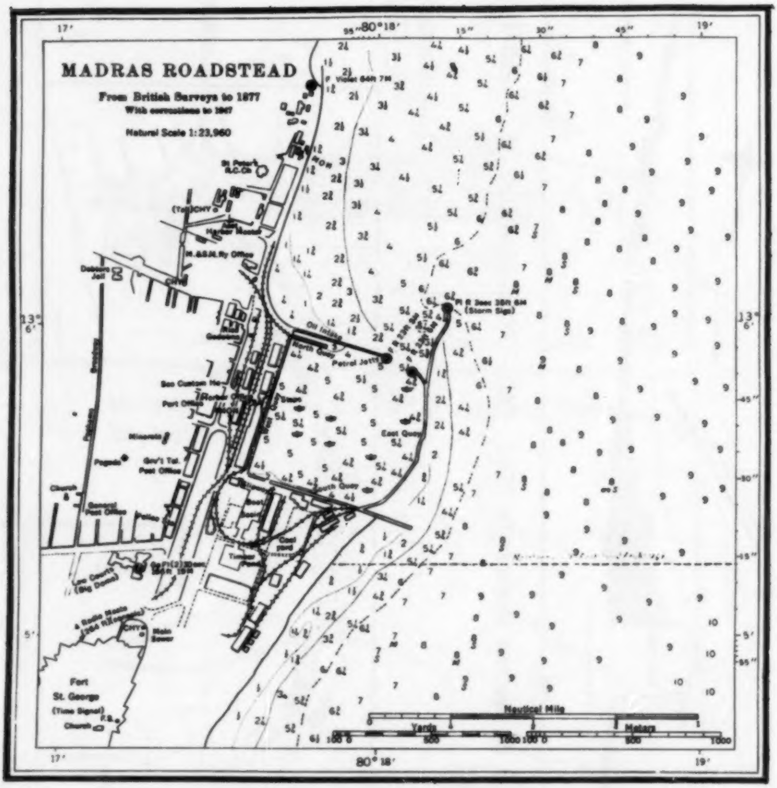


Fig. 11 Madras Harbor, India, in 1947 (from U.S.N. Hydrographic Chart No. 2433)

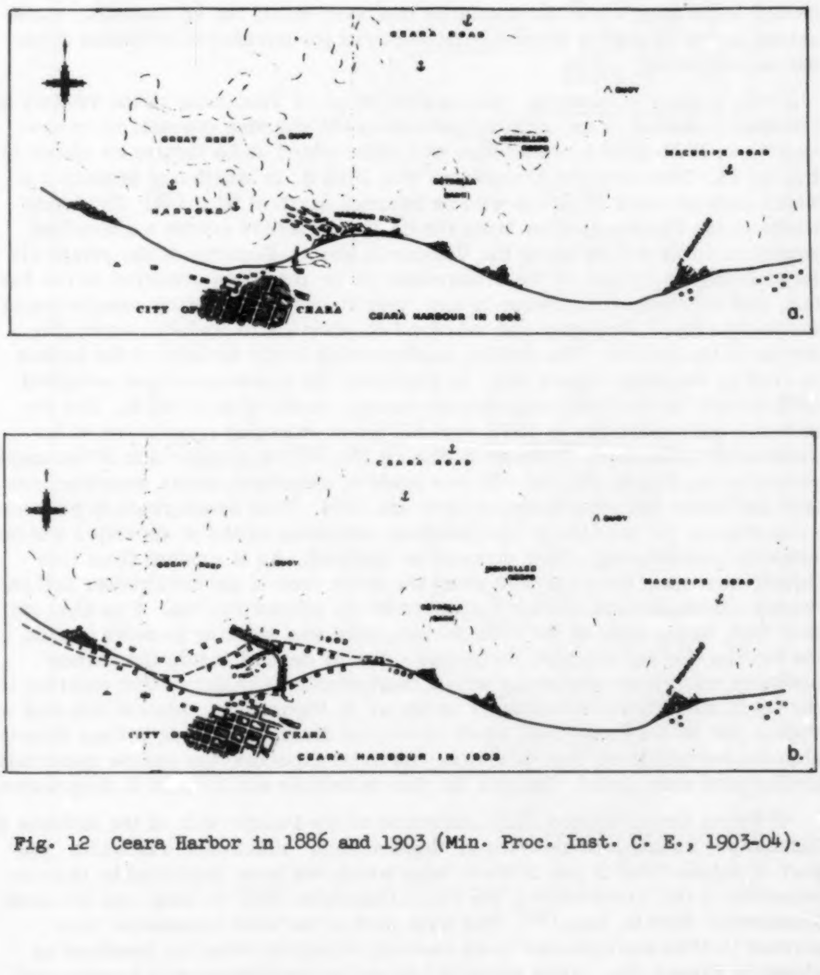


Fig. 12 Ceara Harbor in 1886 and 1903 (Min. Proc. Inst. C. E., 1903-04).

Figure 13 indicates that by 1945 the harbor was practically useless except possibly for small craft. For the relatively high rate of littoral drift which occurs along this coast, as calculated for the nearby harbor at Ponta de Mucuripe, and the relatively small area within Ceara harbor for stockpile purposes, this harbor can be maintained in a condition satisfactory for navigation only by almost continuous dredging.

Ripley[27] stated that the filling of Ceara harbor could have been prevented by the construction of an additional shore-connected breakwater to the west, leaving only a relatively narrow harbor entrance. The fallacy of this design is obvious when one considers that sand would not be moved by wave action in the relatively large depths required for navigation purposes at the harbor entrance.

c) La Guaira, Venezuela. The central coast of Venezuela in the vicinity of Caracas is devoid of any natural protection for shipping operations; consequently in 1885-1888 a breakwater was constructed at La Guaira as shown in Figure 14. This original breakwater was 2050 ft. in length and protected a water area of about 90 acres with an average depth of 30 ft.[28] The trade winds in the Caribbean blow from the east and thereby create a prevailing westward littoral drift along the Venezuela coast. Because of the relatively large depths in the lee of the breakwater tip an accretion occurred in the harbor, and continued to increase in size over the years following completion of the breakwater. Dredging was required at intervals to maintain navigable depths in the harbor. The bottom configuration in the vicinity of the harbor in 1939 is shown in Figure 15a. In 1949-1951 the breakwater was extended 1900 ft. and the harbor dredged to an average depth of about 30 ft. The result of a bottom survey in 1954, several years following completion of the breakwater extension, is shown in Figure 15b.[29] A comparison of the maps presented in Figure 15c and 15b was made to determine areas where accretion and scour occurred between 1939 and 1954. This comparison is presented in Figure 15c with the cross-hatching indicating areas of accretion and the remaining area being either scoured or dredged. As is evident from this figure, accretion has occurred along the outer face of the breakwater and extended a considerable distance seaward of the breakwater tip. It is also evident that, at the time of the 1954 Survey, sand was starting to move around the breakwater tip and into the harbor. A sand deposit inside the harbor probably will eventually occur with a configuration similar to that existing at the tip of the original breakwater as shown in Figure 15a. Also of interest in Figure 15c is the dotted line which shows the division between bottom materials coarser and finer than 0.15 mm. As to be expected, the coarse materials are located near shore, whereas the fine materials are located in deep water.

d) Salina Cruz, Mexico. The shoreline of the Pacific side of the Isthmus of Tehuntepec consists of a series of "hooked bays" which face eastward. The port of Saline Cruz is one of these bays which has been improved by the construction of two breakwaters, the east breakwater 3240 ft. long, and the west breakwater 2164 ft. long.[30] The first part of the west breakwater was started in 1890 and extended in an easterly direction from the headland as shown in Figure 16a. After about 850 ft. of the breakwater was constructed, the harbor filled so rapidly[31] that the alignment was changed to a southerly direction and terminated opposite the tip of the east breakwater to leave an entrance of about 600 ft. in width (Figure 16b). The maintenance of navigable depths in the harbor and in the entrance has been a continuous battle against

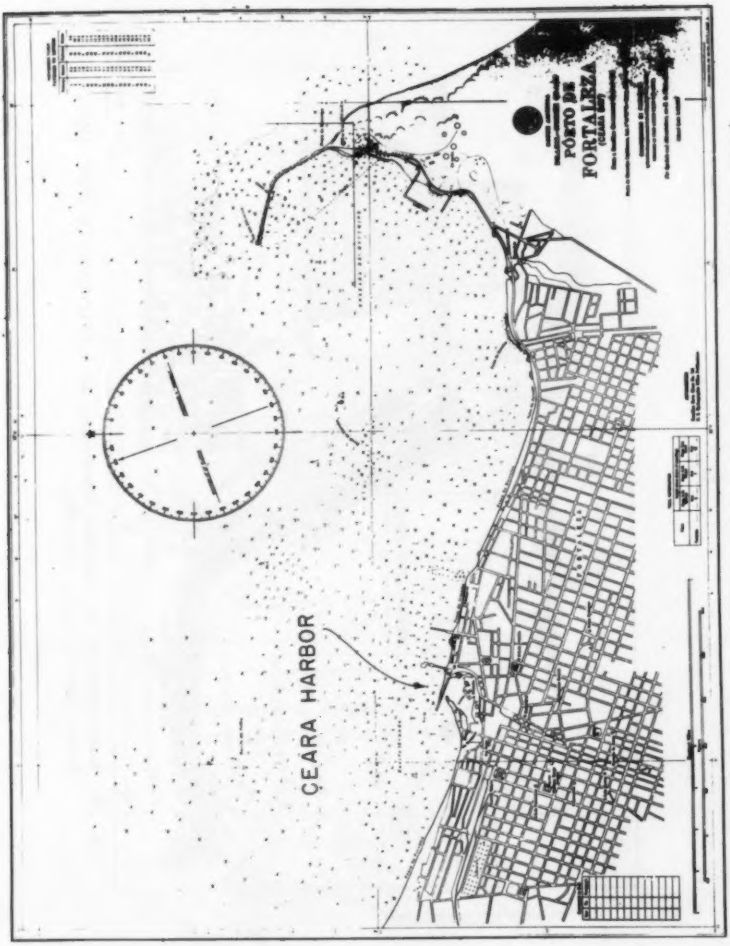


Fig. 13 Fortaleza Harbor, Brazil (U.S. Hydrographic Chart No. 1163).

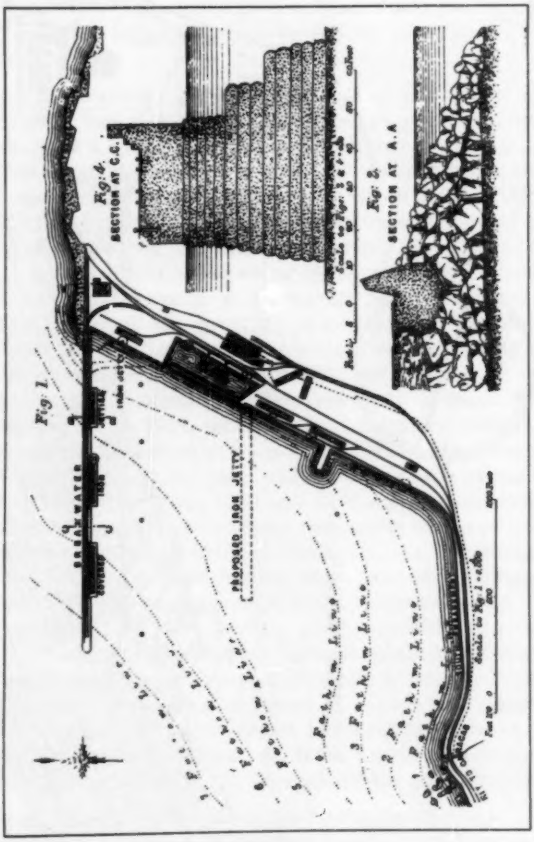


Fig. 14 Breakwater at La Guaira, Venezuela, as constructed in 1885 (Min. Proc. Inst. C. E., 1893-94).

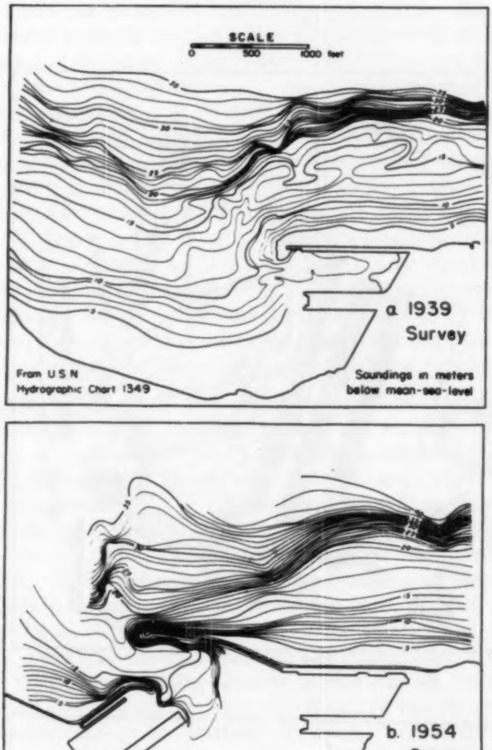
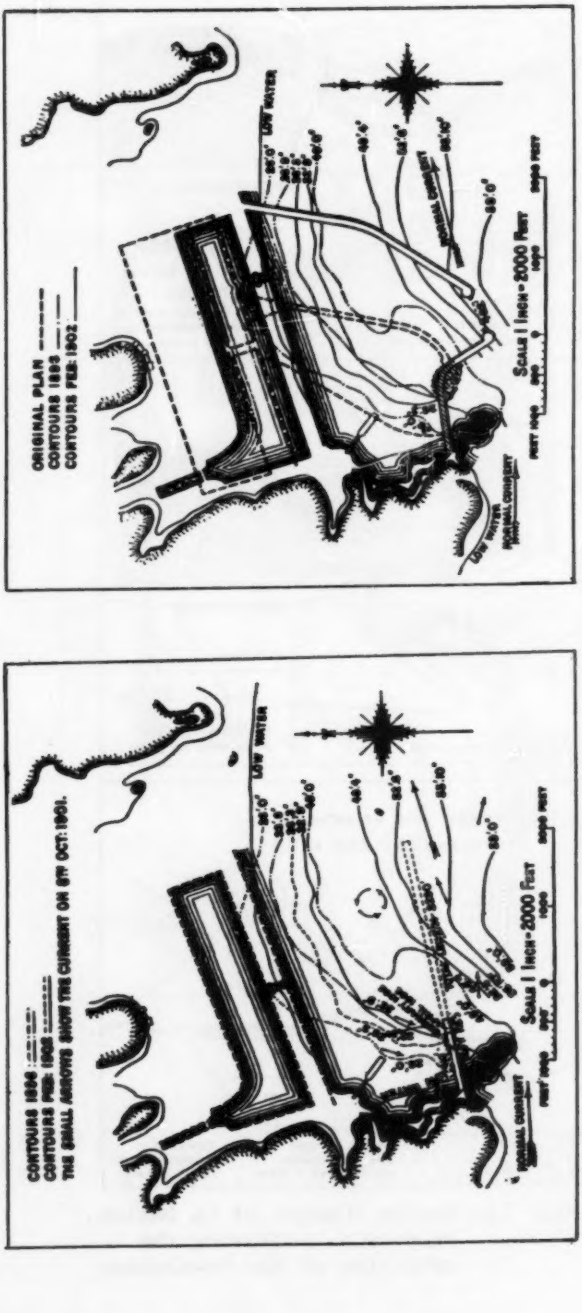


Fig. 15 Bottom changes at La Guaira, Venezuela, following the extension of the breakwater



(a) Initial Breakwater Construction

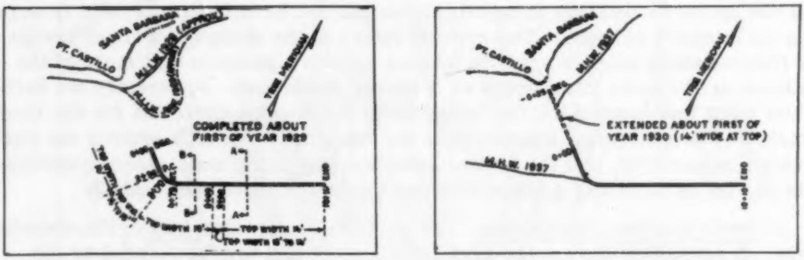
(b) Final Breakwater Construction

Fig. 16 Salina Cruz, Mexico, showing the initial and final breakwater construction (Min. Proc. Inst. C. E., 1903).

the littoral drift since the completion of the breakwaters. The entrance of the harbor is relatively small and the rate of littoral drift so high that stockpiling the material and dredging at infrequent intervals cannot be practiced. Practically continuous dredging therefore has been necessary. In order to by-pass sand at the entrance, a fixed dredge was constructed near the tip of the west breakwater in about 1948. A complete description of this installation has appeared elsewhere and need not be described at this time.(32) It should be mentioned that this installation has not been too successful in serving its intended purpose. The critical factor in the design of a fixed dredge is that the pump intakes must be located in such a position that most of the littoral drift can be intercepted as it moves downcoast. Apparently the Salina Cruz plant was located too far inland from the normal shoreline for the pump intakes to be effective. Inspection of the installation in 1955 showed the fixed dredge inoperative, and the harbor entrance was being maintained at navigable depths by means of a hopper dredge working almost continuously.

e) Santa Barbara, California. The original breakwater was of the detached type. It consisted of an outer arm 1425 ft. long and nearly parallel to the shore, with a short arm about 400 feet long at the western end, with an opening of about 600 ft. between the shore and the inner end (Figure 17a).(33) The breakwater was completed early in 1929, but by the fall of that year shoaling had reached alarming proportions. This shoal consisted of a large sand accumulation along the shoreline and extended seaward in the lee of the breakwater (Figure 17a). The short arm of the breakwater was then extended to the shore with closure being completed about the first of 1930 (Figure 17b). Sand immediately began to accumulate west of the shore arm. By the fall of 1933 the accreting sand has extended almost the entire length of the shore arm (Figure 18). The updrift shore became stable and all littoral material in transit shoreward of the limiting effective depth of the structure moved around it and deposited inside the harbor entrance. To maintain navigable depths in the harbor and to provide nourishment to the easterly beaches it became evident that a program of sand by-passing was necessary. Starting in 1935, the harbor has been dredged every two or three years, as shown in Figure 19 where the total accretion in the harbor since completion of the breakwater is shown. Average daily rates computed for the various periods between harbor surveys also are shown in Figure 19.

To provide detailed information on both the rate of sand transport and the wave characteristics at Santa Barbara, a program of frequent harbor surveys and wave recording was instituted in 1950.(18) A typical example of the growth of the sand deposit in the harbor is shown in Figure 20. A critical examination of the Santa Barbara data reveals that sufficient information is available to establish the average and maximum annual rates of sand movement along that section of the California coast. The data, however, are insufficient to permit the formulation of a general relation between the rate of transport and the wave characteristics. Although the determinations of harbor accretion were made more frequently than in previous years, observations at even more frequent intervals are necessary to determine accurately the actual rate of transport for a given wave condition. Harbor surveys should be made as frequently as the reliability of survey methods permit, but in particular they should be made when the wave characteristics are observed to change appreciably. Frequent observations of harbor accretion and wave conditions, however, are not the only factors to be considered. The effect of tide



(a) Detached Breakwater (b) Breakwater Attached to Shore

Fig. 17 Santa Barbara Breakwater in initial and present form (House Doc. No. 552, 75th Cong., 3d sess.)

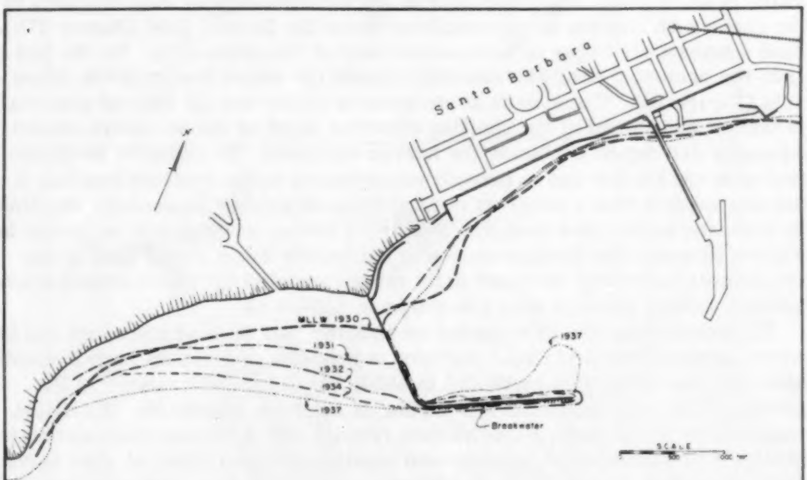


Fig. 18 Shoreline changes upcoast of the Santa Barbara Breakwater

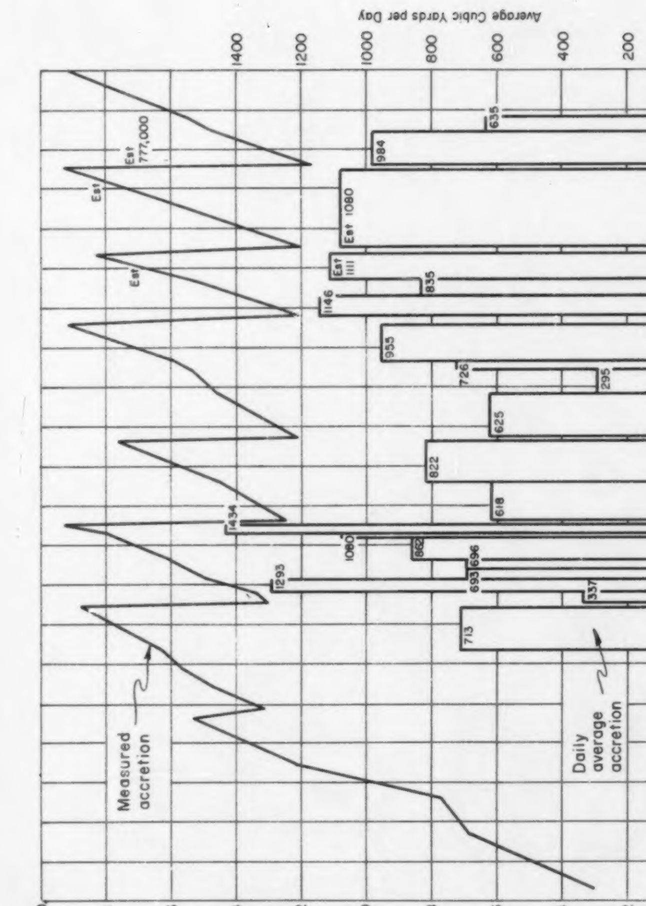


Fig. 19 - Summary of accretion in Santa Barbara Harbor and average rates of littoral transport during periods of harbor survey

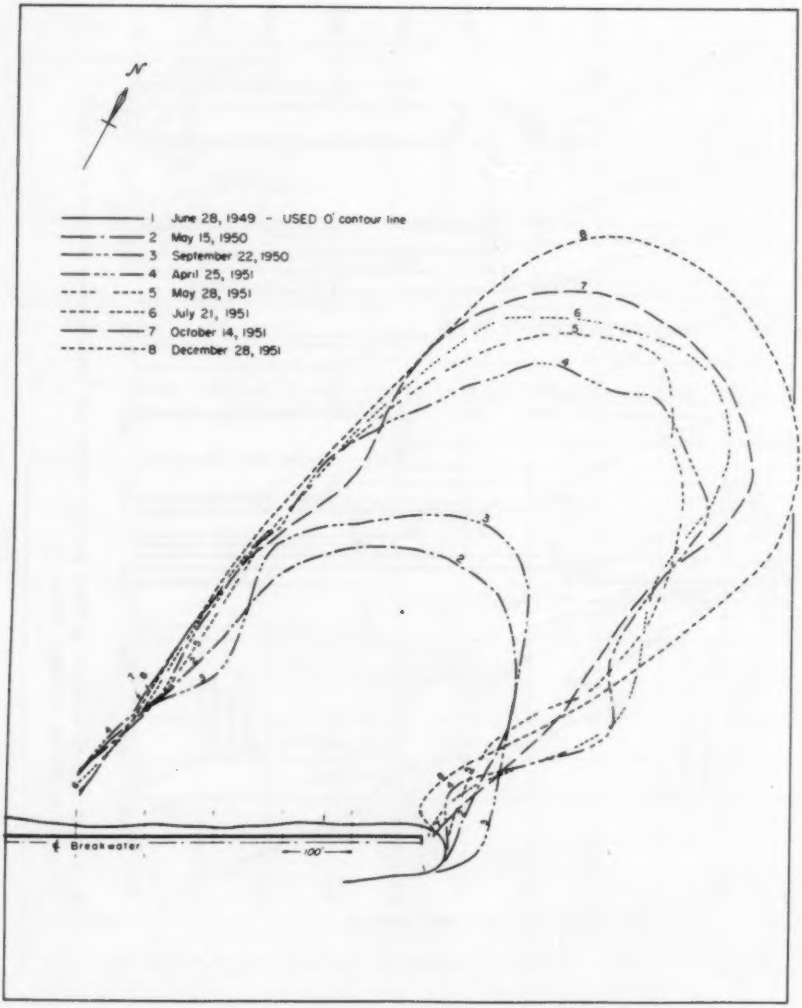


Fig. 20 Growth of sand deposit in Santa Barbara Harbor, California

and the general variability of height, period, and direction of the waves are added complications to the formulation of a general equation for transport.

f) Santa Monica, California. This harbor (Figure 1d) was created in 1934 by the construction of a 2000 ft. length detached breakwater parallel to and about 2000 ft. distant from shore. Immediately following construction of this wave barrier, accretion began to form on the beach in its lee. Within a few years accretion extended seaward enough to act as a groin, and the entire shoreline for some distance upcoast from Santa Monica began to advance in a broad sweeping curve.(34) Downcoast, on the other hand, the shoreline began to recede. Figure 21 shows the position of the mean high water line for various years following construction. As shown in Table I the average rate of accretion upcoast from the breakwater for the period from 1936 to 1940 was 270,000 cu. yds. per year.

g) Fortaleza, Brazil. Approximately three miles east along the coast from Ceara harbor discussed above is the breakwater at Ponta de Mucuripe on which construction was started in 1940 (Figure 13). Prevailing wave action and littoral drift is from the east, and a single shore-connected breakwater approximately 4600 ft. in length has been adequate to provide a protected area for shipping operations. Yearly surveys of the accretion inside the breakwater have been made by the Brazilian Government.(35) A few of these surveys to indicate the nature of this deposition are shown in Figure 22. Calculation of the annual accretion for the period 1946 to 1950, inclusive, gave an average annual rate of littoral drift of 427,000 cubic yards. As shown in Table I, this rate is comparatively high. This harbor has the advantage that a relatively large area inside the breakwater tip is available for stockpile purposes, as compared with the relatively small area available at nearby Ceara harbor. The large stockpile area at Fortaleza, therefore, permits dredging operations to be conducted only at intervals of several years.

h) Port Hueneme, California. This harbor was constructed in 1938-40 and consists of an entrance channel 35 ft. deep and 400 ft. wide, protected by two jetties, 1000 and 1100 ft. long (Figure 23). Since construction of the jetties, accretion has occurred along the upcoast shore. Simultaneously with the upcoast accretion, recession occurred along the downcoast shoreline and in late 1948 extended downcoast a distance of 7 miles.(50) Various attempts at shore protection have been made including placement of fill material and the construction of a stone seawall which extended a distance of about 3000 ft. from the downcoast jetty. In 1954 material was dredged from the upcoast side of the jetties and deposited on the downcoast side to replenish the supply to that reach of the shoreline. Records of the shoreline downcast from Port Hueneme for the 82-year period from 1856 to 1938 showed the shore to be exceptionally stable, thus indicating close balance between supply and loss. The littoral supply passing Point Hueneme prior to construction of the harbor jetties probably was adequate for nourishment of this section of the shore. To insure the future operational condition of Port Hueneme harbor and protect the downcoast shoreline from serious erosion, as well as to provide a separate small-craft harbor, a sand by-passing plan has been developed.(50) As shown in Figure 23, this plan provides for a detached breakwater 2300 ft. long and two harbor entrance jetties about 1400 ft. long each. A sand trap will be provided in the lee of the breakwater by dredging material to a depth of 30 feet. The dredging of 1,000,000 cubic yards of material from the

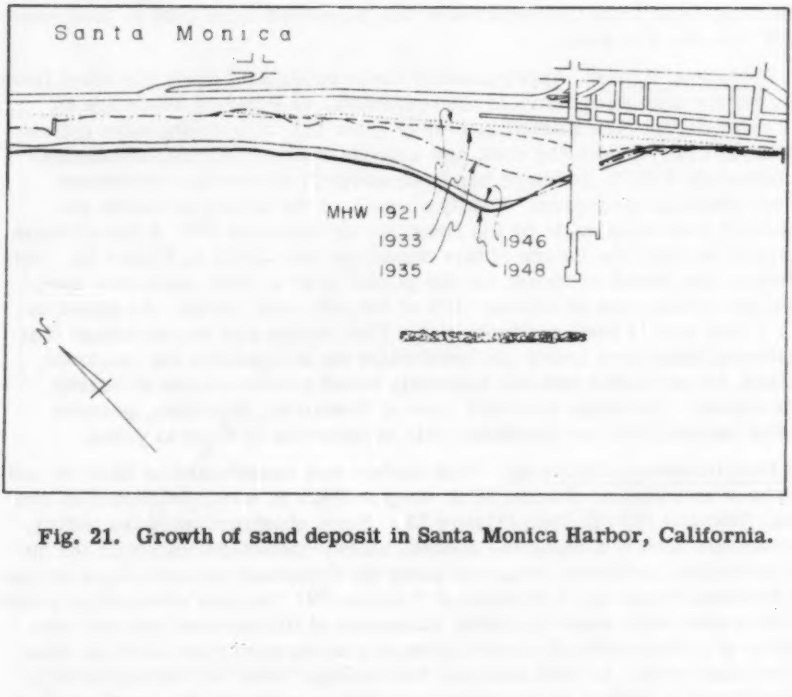


Fig. 21. Growth of sand deposit in Santa Monica Harbor, California.

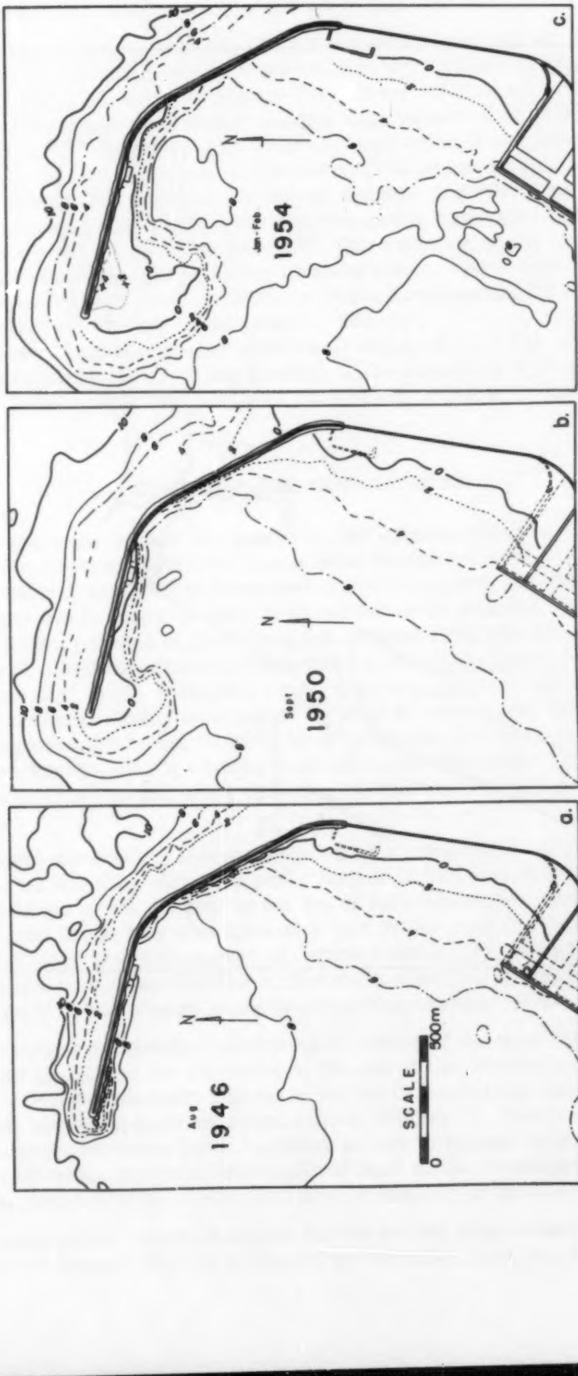


Fig. 22 Growth of sand deposit in Fortaleza Harbor, Brazil. Soundings in meters (from surveys by Departemento Nacional de Portos, Rios e Canais, Rio de Janeiro)

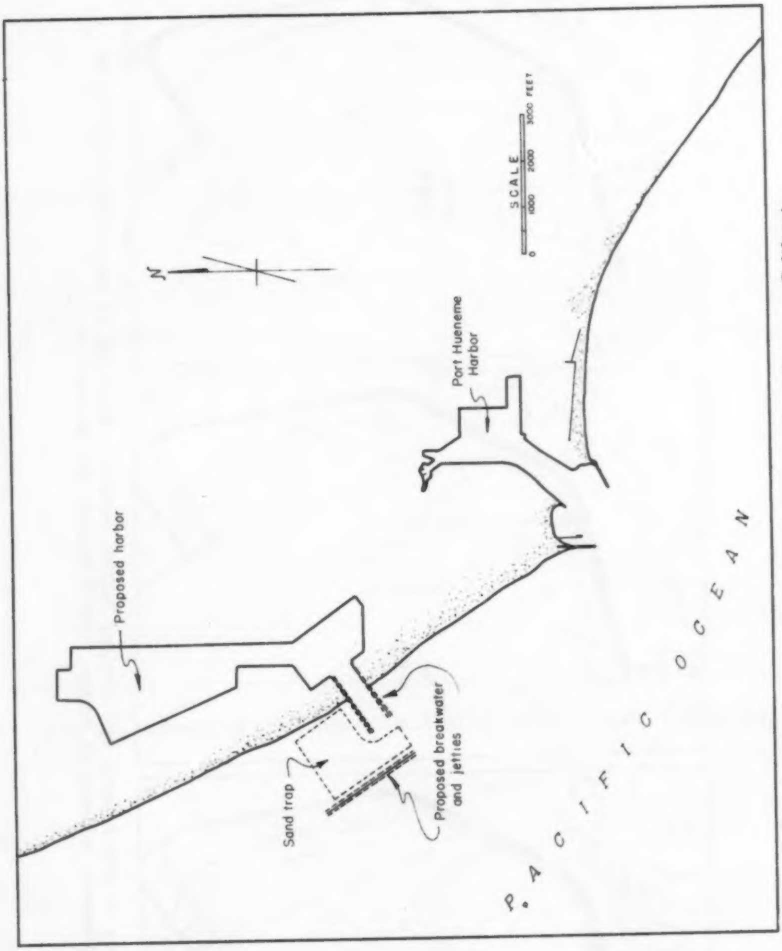


Fig. 23. Present and proposed harbors at Port Hueneme, California.

stockpile area every two years and depositing it downcoast of the Port Hueneme jetties to nournish the downcoast shoreline is expected to be an adequate solution to littoral drift problems at this harbor.

i) Camp Pendleton, California. To form a small boat harbor at this location, two shore-connected jetties were constructed in 1942-1943, and a boat basin was dredged as shown in Figure 24a. Since construction of these structures, progressive shoreline changes have occurred as shown in Figures 24b-d.(62) Studies of the accretion and scour of the adjacent shorelines indicate that the direction of littoral drift is to the south during the winter and spring, and to the north during summer and fall. The resultant littoral movement to the south predominates and is estimated to be approximately 100,000 cubic yards per year.(58) The entrance to the harbor in 1955 was almost completely closed, since no maintenance and dredging had been done for several years prior to 1955. A limited amount of littoral material now passes around the end of the jetties. The stockpile area at this harbor entrance is so small that almost continuous dredging would be required to maintain navigable depths. Some erosion of the shoreline at Oceanside on the downcoast of the harbor has occurred as a result of the accretion of the jetties.

SUMMARY AND CONCLUSIONS

When a shoreline harbor is created by the construction of an artificial littoral barrier, certain basic phenomena must be recognized. The greatest volume of littoral material is transported in the immediate vicinity of the breaker zone where there is sufficient turbulence to maintain the sediment in motion. Along a particular shoreline, but upcoast from the influence of the structures at a harbor entrance, the breaking waves transport littoral material at "capacity" in the relatively small depths of the surf zone. Because of the relatively large depths seaward of the tips of structures, however, the transporting capacity is appreciably less than along the beaches either upcoast or downcoast from the harbor entrance, and deposition of the littoral drift results. The two following common conditions exist which induce deposition:

a) Littoral material is moved to the tip of a breakwater by the turbulence from wave action along the outer face of the structure (Figure 5). Such turbulence is not present in the lee of the breakwater. and the littoral material deposits in the form of a spit in the protected area. In other words, the rate of transport in depths required for navigation purposes is small compared with the surf-zone depths, and deposition occurs at the point of the change in the transporting capacity of the waves.

b) A detached breakwater intercepts a portion of the wave energy with a result that along the shoreline in the lee of the structure the ability of the waves to transport littoral material is much less than on the upcoast shoreline and deposition occurs (Figure 7). Downcoast from the structure the waves have the same ability to transport sand as on the upcoast side. Because the supply of sand to the downcoast beach has been intercepted by the breakwater, recession of the shoreline results.

In planning either a new shoreline harbor or the improvements to an existing harbor on a coast where the littoral transport of sand is a factor, the

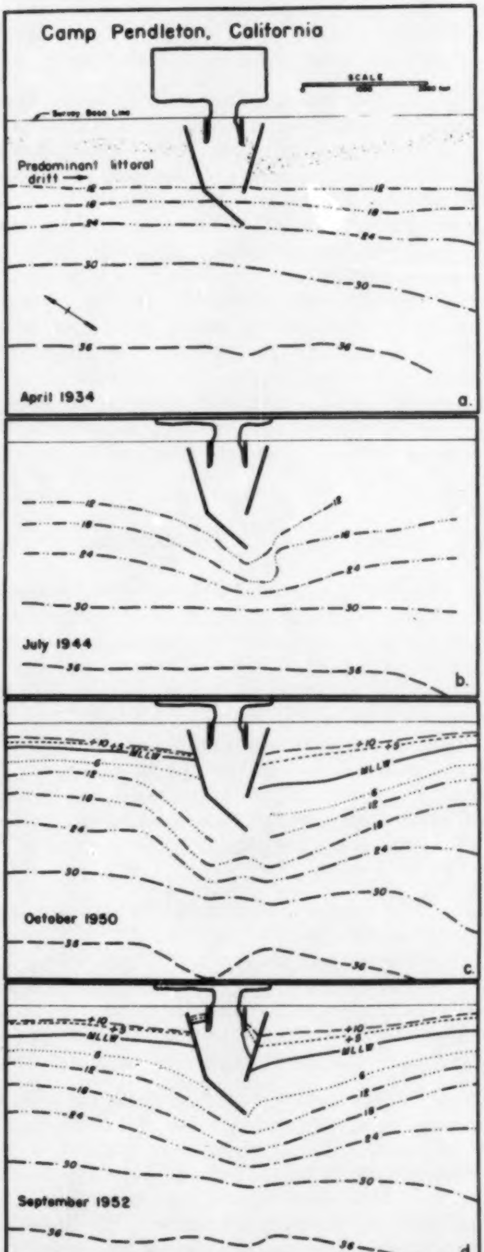


Fig. 24 Shoreline changes at Camp Pendleton Harbor, California

novice in the field is urged to heed the following statements on the need for a thorough study of the local littoral processes and the recognition that any engineering structure will require continued maintenance to insure the proper functioning of the works.

In a paper on the sanding-up of tidal harbors, A. E. Carey(24) in 1904 stated that,

> "A harbor when built has to be defended; moreover, it has to be maintained by dredging, often at a cost relatively high compared with the direct return which it produces as a trade center."

Sir Francis Spring(21) in 1912 stated the following:

> "The chief lesson to be learned from a study of the sand-travel at Madras appears to be that it is absolutely necessary, when an engineer is called on to advise about a work situated on a coast where there is any suspicion of travelling sand, mud, or shingle, that he should be allowed adequate time to make observations, conducted with due precautions, of the directions and causes of the travel; and that if he should arrive at the conclusion that such travel is likely to affect the proposed works in the near or the distant future, he ought at least to acquaint his employers with what he conceives to be the broad general facts of the case and their probably financial effects, whether on the proposed works or on other works or interests."

Sir Francis Spring stated in the above paper that the movement of the sand on the east coast of India threatened to overwhelm Madras Harbor unless adequate precautions were taken to counter it. In a discussion of the paper J. M. Dobson(36) stated:

> ". . . . . it was an established fact that, wherever harbors were built on sandy coasts, such harbors must naturally be maintained either by dredging, or by other means adapted to counteract the sand-accretion; and it was almost impossible to imagine why it should ever have been assumed that Madras Harbor, on a coast where the sand was always on the move, should prove an exception to this rule."

In the Preface to the Proceedings of the First Conference on Coastal Engineering, 1950, M. P. O'Brien cautioned that the design of coastal works involves many criteria which are foreign to other phases of civil engineering, and that:

> "Along the coastlines of the world, numerous engineering works in various states of disintegration testify to the futility and wastefulness of disregarding the tremendous destructive forces of the sea. Far worse than the destruction of insubstantial coastal works has been the damage to adjacent shorelines caused by structures planned in ignorance of, and occasionally in disregard of, the shoreline processes operative in the area."

ACKNOWLEDGMENTS

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CONTENTS

DISCUSSION
(Proc. Paper 1226)

Discussion of
"IMPROVING THE GULF INTRACOASTAL CANAL IN TEXAS"

by Willard P. McCrone
(Proc. Paper 967)

E. D. BUTCHER.[1]—As a barge operator, the writer is naturally very interested in improvements in the Gulf Intracoastal Waterway. Likewise, the writer is certain that there are improvements to be made in this waterway which will yield a very high benefit-to-cost ratio. Furthermore, the calculations which Mr. McCrone shows in his paper confirm the writer's experience as to the effect of deeper channels in increasing the speed of a given tow and reducing the ton-mile cost of the transportation produced by the tow unit. Nevertheless it is not thought that deepening should be the next step in the improvement of the Gulf Intracoastal Waterway. Widening will increase efficiency of barge transportation to a much greater extent that deepening.

The writer views the Gulf Intracoastal Waterway as only a part of the Mississippi River System of barge transportation. It is therefore fundamental that the first goal of a program of improvement of this waterway should be to make it useable to the efficient big tow which operates on the Mississippi, Ohio, Illinois, and Tennessee Rivers. Full utility of this waterway can never be realized until it is no longer a narrow-gage tributary to a wide-gage national river system.

It is regrettable that the barge operators and the Corps of Engineers did not foresee in the beginning nor realize in recent years that this waterway was not an independent facility but was merely an extention of the national river system. There are a number of locks, gates, bridges, and other structures which certainly should have been built to river dimensions; others, such as the Old River locks, are now being considered. It is most important that no other undersized improvement be permitted. The standard river lock width is 110 ft.; one hundred years from now it will probably still be the standard width. Boat and barge design will certainly be controlled by this width in the future as it has been in the past. This width then should be a datum point in considering all projects.

Fortunately, it can be shown that a width of 110 feet is a very efficient width. A tow with a width of approximately 105 feet and a length of 1,200 feet or less, (including the towboat) is the design tow for the locked rivers. This tow at a draft of somewhat less than 9 feet has a cargo capacity of approximately 23,000 tons. It is certainly big enough to give peak efficiency in operation. Such tows are in operation and have demonstrated that they are able to cope with river conditions generally. Almost without exception barges have been built and are being built with widths of 26 feet, 35 feet, 50 feet, and 52-1/2 feet—multiples of which will fit into 110 feet locks.

Many 50-feet wide barges and tugs and towboats of from 700 horsepower to 900 horsepower operate on the waterway. This equipment was built to this design largely because of limitation of width on the Gulf Intracoastal

1. Executive Vice-Pres., Commercial Transport Corp., Houston, Texas.

Waterway; a desire for larger tows for greater efficiency could not be fulfilled. With a channel width of 125 feet, a barge width of 50 feet is certainly wide enough. Two of these barges totaling 480 feet in length and carrying 5,000 tons of cargo are near the maximum tow which could be operated on the waterway in adverse, as well as good, weather. Considerably larger tows have been operated on the waterway but they are unable to operate in high winds when empty; they take up both sides of the channel when navigating the slightest bend and, in general, navigate by "worrying" the tow through. Under these circumstances a 5,000 ton tow with a tug or towboat of from 700 horsepower to 900 horsepower is the most economical tow on the waterway from a ton-mile cost. These tows are fairly efficient but they are inflexible. There is a constant demand for towing odd barges to and from the river; most of these barges are 195 feet by 35 feet. They cannot be added to the regular tows because of channel restrictions. There is much odd-barge movement and a need for more. With the inflexibility of tows in regard to adding an additional barge, however, service to the odd-barge shipper is inferior and costly; only a wider channel can improve this situation.

The present channel width of 125 feet restricts the use of the most popular barge size on the river system, the 195 ft. by 35 ft. "jumbo" barge. For efficient, safe navigation three of these barges make a maximum tow. In general commodity movements, for which they are ordinarily used, the barges are usually loaded with only from 500 to 1,000 tons. A tow of three of these barges, therefore, carries only from 1,500 tons to 3,000 tons. The cost of this type of movement precludes rate reductions which would attract additional traffic to the benefit of both carrier and shipper.

The writer's recommendation for a minimum project for the improvement of the Gulf Intracoastal Waterway is to widen it to 250 feet from Morgan City to Bolivar Roads. Since the opening of the Atchafalaya River, this section carries the heaviest tonnage and needs the big tow much more than other sections. With three-to-one side slopes, widening from 125 ft. to 250 ft. will increase the cross section by 1,500 sq. ft. as compared with 1,290 sq. ft. for deepening to 18 ft. If the cost of dredging for the additional channel cross section were prohibitive, the width could be reduced to 225 ft., and two big tows could still pass with from 5 ft. to 20 ft. of clearance. The comparison in operating costs between the big tow and tows now operating on the waterway would be even more striking than the increase in speed which results from a deeper channel. In general, the resistance of a tow varies as the square of the speed and as the first power of the displacement. Also, cost of operation varies as a fractional power of horsepower. These factors result in much lower cost per ton-mile as the size of the tow is increased.

There are several other factors which also need to be considered. In the writer's experience high-speed operation close to a bank is very dangerous in that the tow has a tendency for the bow to run for the middle of the stream and the stern for the bank, presenting a serious collision hazard in passing situations. Second, the experience on wide, shallow rivers indicates that width is as effective as depth in reducing tow resistance. It is possible to travel at from 8 miles to 8.5 miles per hour in 12 ft. of water with a 1,500 horsepower boat and a 7,500 ton tow where the river is from 250 ft. to 300 ft. wide. Third, in the 125 ft. sections of the Gulf Intracoastal Waterway, approximately 1,000 horsepower is the maximum power that can be used. More power only causes the boat to "squat" without increasing its speed appreciably. This effect is not observed in the wide, shallow rivers except with

speeds approximating 9 miles per hour and in cases of depths of less than 9 feet.

In summary, a wide channel has decreased resistance to the tow and enables the use of greater power in the towing vessel. In addition, a wider channel would allow the most efficient tows of the river system to navigate under reasonable conditions to the ultimate destination of most of their barges. Flexibility of tow size and make-up is possible only in a wider channel, and only with flexible tows can the odd-barge shipper be properly served and at reasonable rates. For these reasons the writer urges that primary consideration be given to improvement of the Gulf Intracoastal Waterway by widening rather than deepening.

BAILEY T. DE BARDELEBEN.[2]—From a study of Mr. Butcher's discussion, the writer must agree wholeheartedly with a number of his statements. A few of them, however, are merely his opinions and with two or three the writer disagrees.

The writer is certainly in favor of improving the Gulf Intracoastal Waterway, whether it be by deepening or widening, or both. It cannot be agreed, however, that widening will have the same or a better effect than deepening insofar as the efficient movement of the tow through the water is concerned. Certainly the matter of tows passing each other in the narrow waterway is a problem that will be greatly relieved by widening. Also, in widening, the present bends will be eased to a great extent and in some instances entirely eliminated.

The writer does not look upon the Gulf Intracoastal Waterway as a part of the Mississippi River System of barge transportation any more than one considers the Missouri Waterway or the Illinois Waterway as parts of the Mississippi River System. It does not appear feasible to widen the Gulf Intracoastal Waterway sufficiently to approach the width necessary to allow tows of the same size to traverse its waters as on the Mississippi River. The special tows on the Mississippi River are now capable of traversing the Gulf Intracoastal Waterway, but the large common-carrier tows with the odd barges picked up and dropped at various places would require a canal several times wider than the 300 ft. mentioned.

It is indeed regrettable that the foresight of those developing the Gulf Intracoastal Waterway was not sufficient to plan a waterway of much larger dimensions. This is a regret on practically all waterways and at the present time a very serious one as to the locks on the Ohio River and Warrior River, not to mention the bridges on the Illinois Waterway. It is agreed that the standard Ohio River Lock of 110 ft. will most likely become the standard lock width in the next one hundred years. It is also agreed that the present design of boats and barges will operate very efficiently in waterways having locks of 110 ft. by 1200 ft.

The writer is associated with a company that was among the first canal carriers to build towboats for use in the canal. It has been proved that a towboat of from 1000 hp to 1200 hp is the most efficient in the Gulf Intracoastal Waterway. These twin-screw towboats of 1200 hp can operate efficiently tows of 1000 ft. long (not including the towboat) and 52 ft. wide, carrying approximately 8000 tons from New Orleans to any of the western ports. These tows can handle very efficiently five 195 ft. by 35 ft. barges; the cargo

2. Pres., Coyle Lines Inc., New Orleans, La.

capacity of these barges is fully utilized in most instances. Seldom will five of these barges carry as little as 5000 tons. Three large barges, 45 ft. or 48 ft. wide by from 230 ft. to 300 ft. long carrying approximately 2500 tons each or a total cargo of 7500 tons are often moved; this tow is a maximum of 900 ft. in length. To counter the statement that canal tows are inflexible, mixed tows of barges 175 ft. by 26 ft., 195 ft. by 35 ft., 132 ft. by 35 ft., and 300 ft. by 48 ft. move through the canal regularly dropping off barges at various ports and picking up other barges for movement beyond. The service to odd-barge shippers is satisfactory and efficient and it is not a costly operation; it is not true that the service to these shippers is inferior. A wider canal will improve the situation as far as ability to pass tows without damage is concerned, but, also, a deeper channel would allow one to move tows through the canal somewhat faster than at present. A maximum of three "jumbo" barges is not the most efficient and safest tow for barges of that size. As stated previously, five of these barges are often moved in one tow in 3 1/2 days from New Orleans to Houston with about 5500 tons of cargo.

Integrated tows 880 ft. long and 50 ft. wide are presently operating on the Mississippi River between Port Sulphur and St. Louis. These barges carry 7500 tons of cargo and are pushed by a 3200 hp towboat. The tow moves along at a good speed until it passes Baton Rouge at which point the drop is almost immediately noticeable; and by the time the tow reaches Natchez, it has lost several miles per hour in speed. This can only be attributable to the depth of the water and not to the width. A drop in speed is noticed in shallow rivers where the depth is between 9 ft. and 12 ft.

As stated previously, it is agreed that additional width would aid in the flexibility of tow sizes, but flexible tows with varying barge sizes can now be operated, giving the odd-barge shipper proper service at reasonable rates.

WILLARD P. MC CRONE,[3] M. ASCE.—Apparently, Mr. Butcher is not entirely aware of the procedure which the Corps of Engineers must follow in studying and approving an improvement such as the Intracoastal Waterway. The paper outlines, in general, the steps which are taken, and the final plan of improvement to be adopted depends strictly on the existence of a favorable ratio of benefits to cost. The original Intracoastal Waterway Project was justified on an estimated annual tonnage of 5,000,000. Of course this project has succeeded beyond the original promoters' fondest hopes, but this could not be foreseen at the time the project was originally authorized. At the time of the original survey all the barge and shipping companies were contacted and the sum of their estimates comprised this estimate of 5,000,000 tons. Based on that, the improvement which was provided was the maximum which could be supported at that time. It was realized that the waterway was an extension of the national river system, and should be constructed with the maximum possible dimensions. However, the estimated tonnage of traffic did not support the cost of maximum development during the initial construction. It would have been wise to secure a greater width of right-of-way initially, since it could have been accomplished then at a small increase in cost.

Mr. Butcher is probably also aware that the construction of a project of this type is a joint procedure with the local interests providing the necessary rights-of-way and other real estate items for the project. One of the big

3. Col., Dist. Engr., Galveston Dist., Corps of Engrs., U. S. Dept. of the Army, Galveston, Tex.

problems in any additional expansion of the waterway is the difficulty in securing interested local authorities who will purchase the required additional land. Funds are always limited, and unless some direct and appreciable local benefit is apparent, local interests understandably are reluctant to spend any more than is immediately necessary. It is for this reason, primarily, that the Corps of Engineers has studied increase in depth as well as increase in width. Admittedly, the overall cross section is the primary factor as long as the depth does not get less than about 9 ft. Cost, however, in widening as opposed to deepening, does increase rapidly. In Table 8 the cost of an 18-ft. by 250-ft. channel as compared to an 18-ft. by 125-ft. channel is more than three times as much. The cost of a 12-ft. by 250-ft. channel would not be so high, of course, but still it would be appreciably higher than the 18-ft. by 125-ft. width of channel because of the added rights-of-way requirements. As pointed out in the paper, increasing the depth to 18 ft. with a bottom width of 125 ft. will provide a channel width of 161 ft. at the present 12-ft. depth. This 161-ft. width will permit the use of two jumbo barges, side by side. Because this will be possible, a tow can probably be made up of eight of these barges with a carrying capacity of from 4,000 tons to 8,000 tons. This tow width of 70 ft. will enable tows to pass through the eleven limiting bridges in Louisiana, the floodgates at the Brazos River, and the locks at the Colorado River which have horizontal clearances of 75 ft. Some modifications would be necessary to three of the locks in Louisiana to accommodate this width. Although this is certainly much less than a 23,000-ton tow, it still is a major improvement over the present capacity of the waterway.

DIVISION ACTIVITIES
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

April, 1957

COMMITTEE ON SESSION PROGRAMS HAS ACTIVE YEAR

During 1956, the Division Committee on Session Programs, together with other Division Committees, arranged for the presentation of 30 papers at three conventions.

Dallas Convention - February 1956. The Division sponsored a session of four papers and co-sponsored, with the Hydraulics Division, a second session of four papers. The program was arranged by Mr. Donald H. McCoskey. Attendance ranged from 30 to 100.

Knoxville Convention - June 1956. The division sponsored two sessions of four papers each. Attendance at the sessions, arranged by Col. Gilbert M. Dorland, ranged from 50 to 80.

Pittsburgh Convention - October 1956. Participation in this Convention by the Division consisted of two technical sessions developed by the Committee on Navigation and Flood Control Facilities and the Committee on Ports and Harbors, of which Mr. C. F. MacNish and Frank W. Herring are chairmen, and two technical sessions co-sponsored by the Waterways and Harbors and Hydraulics Divisions. One of these sessions was developed by the Coastal Engineering Committee, of which Mr. H. O. Eaton is chairman. Thw Waterways and Harbors Division co-sponsored with the Construction and Hydraulics Division a luncheon featuring an address by Major General Emerson C. Itschner, Chief of Engineers, U. S. Army, on the subject: "Tomorrow's Demands will be Greater." The Division sponsored a field trip by boat of the Pittsburgh Industrial Waterfront. The attendance of the technical sessions ranged from 40 to 80, and over 200 attended the luncheon.

In February 1957, the Committee arranged for two sessions and a luncheon at the Jackson Convention. Eight papers were presented at the technical sessions, with attendance ranging from 75 to 100. About 250 attended the luncheon, which was addressed by Gen. Hardin, President of the Mississippi River Commission. Special mention was made at the Executive Committee meeting during the convention of the excellent program.

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RESEARCH IN FIELDS OF WATERWAYS AND HARBORS

The Committee on Research of the Waterways and Harbors Division has prepared the following list of publications which give a comprehensive summary of all U. S. research in the fields of waterways and harbors.

Engineering College Research Council of the American Society for Engineering Education. Review of Current Research and Directory of Member Institutions, 1955, New York, May 1955.

Engineering Foundation. Annual Report, October 1, 1954 to September 30, 1955. New York, Engineering Societies Building, 20 West Thirty-Ninth Street, October 1955.

U. S. National Bureau of Standards. Hydraulic Research in the United States, 1956. Miscellaneous Publication No. 218, 1956. (For sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C.

Waterways Experiment Station. Annual Summary of Investigations in Hydraulics, Soil Mechanics, Pavements, and Concrete for Calendar Year 1955. Vicksburg, June 1956.

SUBJECTS FOR FUTURE RESEARCH PROJECTS

Among the matters discussed at the Division's Committee on Research November 1956 meeting were possible subjects for future research. Some of the subjects considered were:

1. The adequacy of present port facilities and services to accommodate nuclear powered ships.
2. Substructures for piers and wharves—types of construction and related use criteria.
3. Deepening of ships' berths. Effects on stability of existing pier and wharf substructures.
4. Trends in ship sizes, including greatly increased drafts, and effects on channel and harbor facilities.
5. Wave action caused by vessel movement in restricted waterways.
6. Mooring of vessels.

PLANS FOR FUTURE CONVENTIONS PROCEEDING RAPIDLY

For the June 1957 meeting to be held in Buffalo, a total of five sessions is tentatively proposed, four of which will be joint sessions with the Hydraulics Division. There will be a total of about 19 papers. The Committee on Navigation and Flood Control Facilities, the Committee on Research and the Coastal Engineering Committee of the Division, and the Research and Design Committee of the Hydraulics Division are actively engaged in firming up specific titles and authors. The papers will include coverage of a number of the St. Lawrence Seaway locks and channels, as well as model studies in the U. S. and Canada. A number of Canadian authors will participate in the program. There will also be papers on Great Lakes Harbors, Diversion from

the Great Lakes, and Regulation of Lake Ontario by the International St. Lawrence River Power Project.

At the New York Annual Meeting in October, the Division will hold two sessions with a total of seven papers, three by the Committee on Coastal Engineering. Two by the Committee on Navigation and Flood Control Facilities and two sponsored by The Bureau of Yards and Docks, U. S. Navy. In addition, there will be a full day session at Princeton University following the meeting in New York. The Division would participate with the Structural Division and furnish two papers on wave forces in a joint session. The latter papers will be prepared under the direction of the Committee on Coastal Engineering.

COMMITTEE ON PUBLICATIONS PROCESSES 41 PAPERS

In the year ending September 30, 1956, the Division Committee on Publications handled 41 papers, of which 8 were carried over from the previous year and 33 were recommended for publication, and four Division Journals were issued. Of the remaining 9 papers, 7 were in the process of review or revision and 2 were withdrawn.

NEW APPOINTMENT TO COMMITTEE ON PUBLICATION

The Division Committee on Publications has been strengthened by the recent addition of Mr. James W. Dunham, Senior Harbor and Port Engineer, Ralph M. Parsons Company, Los Angeles, California. By this addition, the committee now has representation from the West Coast to balance that of Mr. Buckley of the Department of Marine and Aviation, City of New York, and Mr. Caldwell and Mr. Hall of the Beach Erosion Board of the Corps of Engineers, Washington, D. C. Ellsworth I. Davis, recently appointed Chairman of the Committee, has been promoted to Brigadier General. General Davis is located at Hq., U.S.A. Engineer Center, Fort Belvoir, Virginia.

NEW TASK COMMITTEES FORMED

Two new task committees have been formed under the Division Committee on Coastal Engineering. These committees will consider groins for shore protection and sand by-passing systems.

NEW APPOINTMENT TO COMMITTEE ON NAVIGATION AND FLOOD CONTROL FACILITIES

Mr. M. V. Meland has been appointed as Secretary of the Committee on Navigation and Flood Control Facilities, replacing Mr. W. E. Kindel who was unable to continue his duties due to a change of address. (This Committee was formerly the Committee on Design, Construction and Operation of Navigation and Flood Control Locks and Dams.)

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

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APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941 (SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958 (SA2), 959(PO2), 960(PO2).

MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW3), 968(WW3), 969 (WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2).

JUNE: 990(PO3), 991(PO3), 992(PO3), 993(PO3), 994(PO3), 995(PO3), 996(PO3), 997(PO3), 998 (SA3), 999(SA3), 1000(SA3), 1001(SA3), 1002(SA3), 1003(SA3)[c], 1004(HY3), 1005(HY3), 1006 (HY3), 1007(HY3), 1008 (HY3), 1009 (HY3), 1010 (HY3)[c], 1011 (PO3)[c], 1012 (SA3), 1013 (SA3), 1014(SA3), 1015(HY3), 1016(SA3), 1017(PO3), 1018(PO3).

JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023(ST4), 1024(ST4)[c], 1025(SM3), 1026 (SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048 (SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4), 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068 (WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092 (HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103 (IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c], 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127 (SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

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JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158 (EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)[c], 1193 (PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203 (SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218 (SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226 (WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

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c. Discussion of several papers, grouped by Divisions.